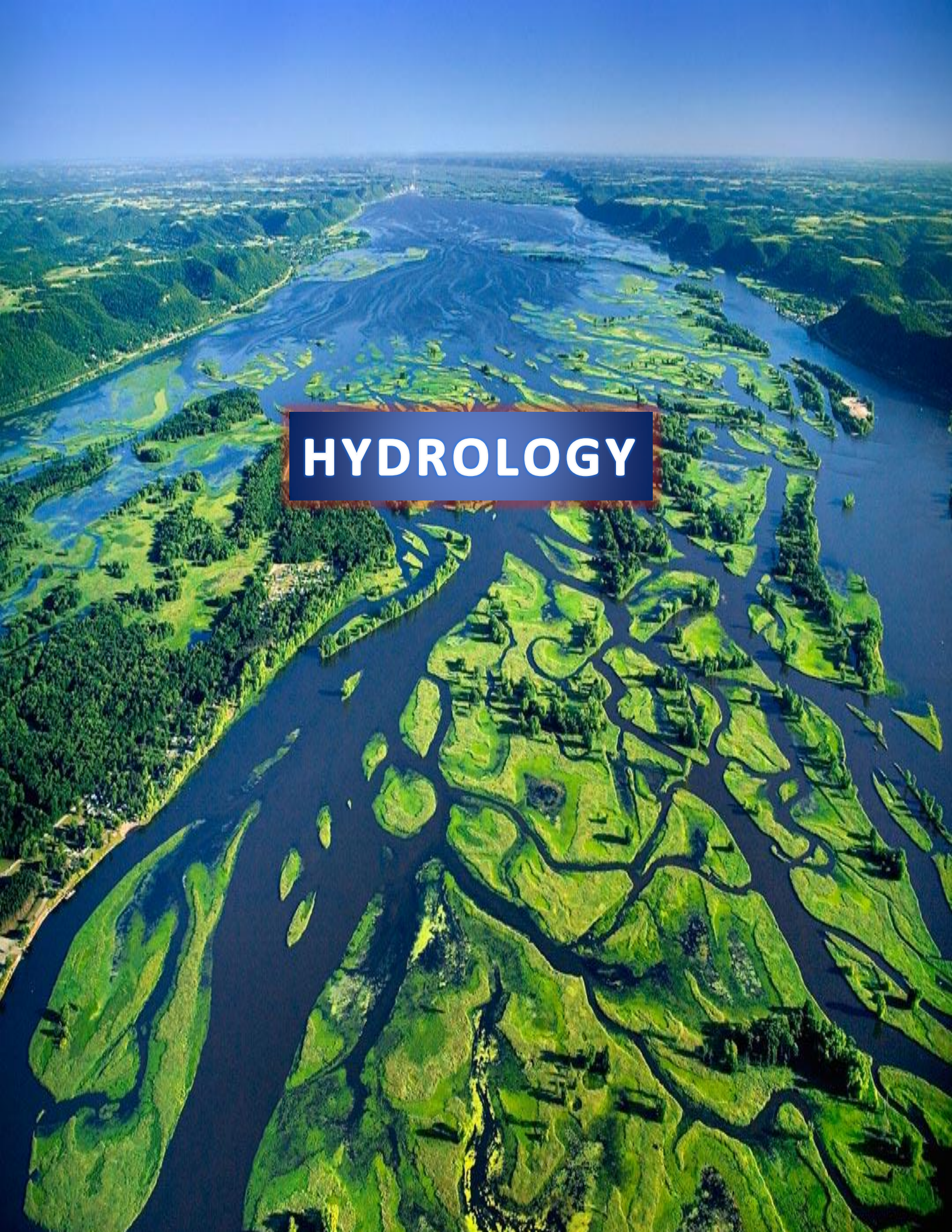
HYDROLOGY

MD FARHAD HOSSAIN

CE 15, 1500045

"Are those who have knowledge and those who have no knowledge alike? Only the men of understanding are mindful. " (Quran, 39:9)

Special THANKS to-

My friend

SAYEM AHAMEED

CE 15, 1500119

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Introduction

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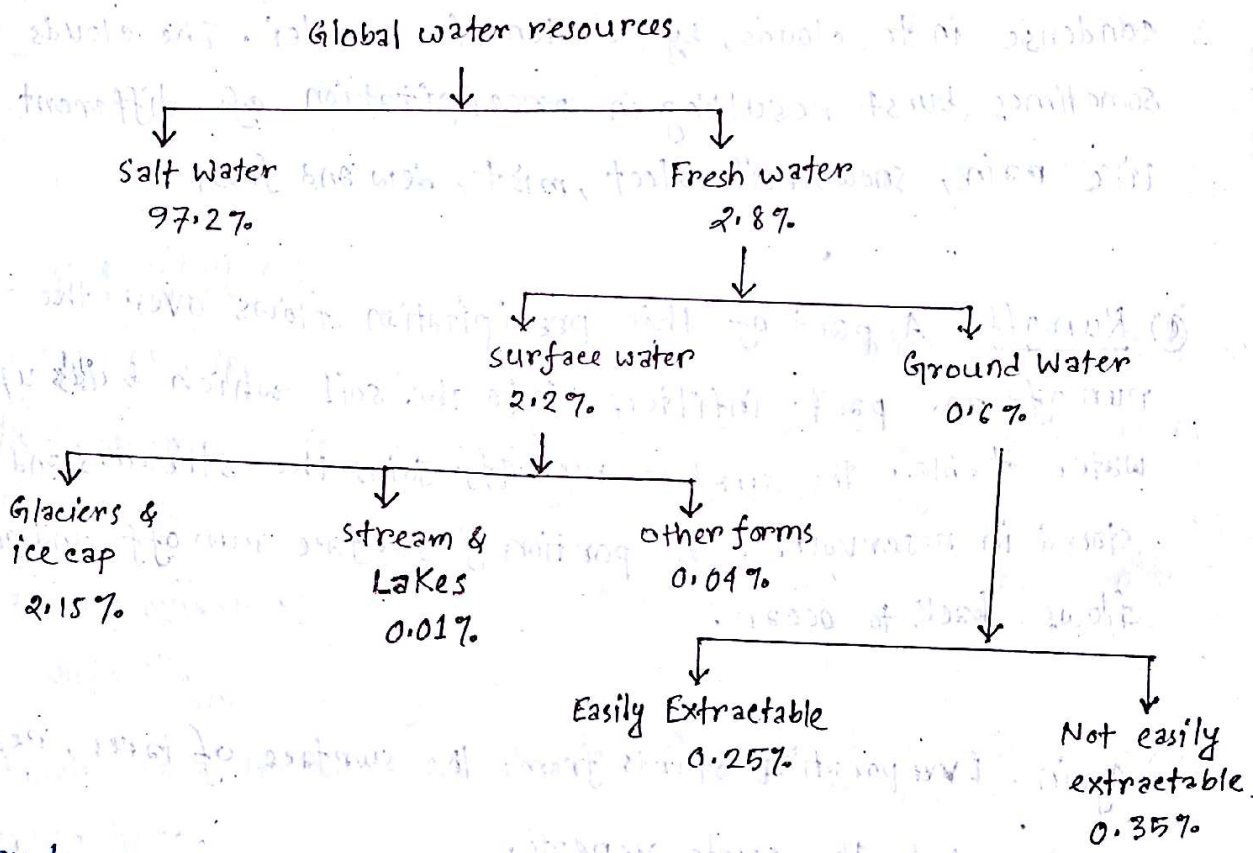
 # Hydrology: Hydrology is a branch of earth science. The word 'Hydrology' is derived from Greek words 'hydor' meaning 'water' and 'logos' meaning 'scientific study'.

Hydrology is the scientific study of movement, distribution, and quality of water on earth and other planets, including water cycle, water resources and environmental watershed sustainability.

 # World's water resources:



 # Hydrologic cycle: 16, 11, 10

Hydrologic cycle is the water transfer cycle, which occurs continuously in nature.

The three important phases of hydrologic cycle are:

(a) Evaporation and Evapotranspiration (b) Precipitation (c) Run off

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(a) Evaporation and Evapotranspiration: The globe has one-third land and two-third ocean. The sun, which drives the water cycle, heats water in oceans and seas. water evaporates water vapor into the air. Evaporation from the surfaces of ponds, lakes, reservoirs, ocean etc. and transpiration from surface vegetation i.e. from plant leaves of cropped land and forests, etc. takes place.

(b) Precipitation: As altitude increases, air pressure decreases and the temperature drops. The lower temperature causes water vapor to condense in to clouds, by condensation nuclei. The clouds melt and sometimes burst resulting in precipitation of different forms like rain, snow, hail, sleet, mist, dew and frost.

(c) Run off: A part of this precipitation flows over the land is called run off and part infilters into the soil which builds up the ground water table. The surface run off joins the streams and the water is stored in reservoirs. A portion of surface run off and ground water flows back to ocean.

Again, Evaporation starts from the surface of lakes, reservoirs and ocean. And the cycle repeats.

Net sketch of Hydrologic cycle: 16,11,10

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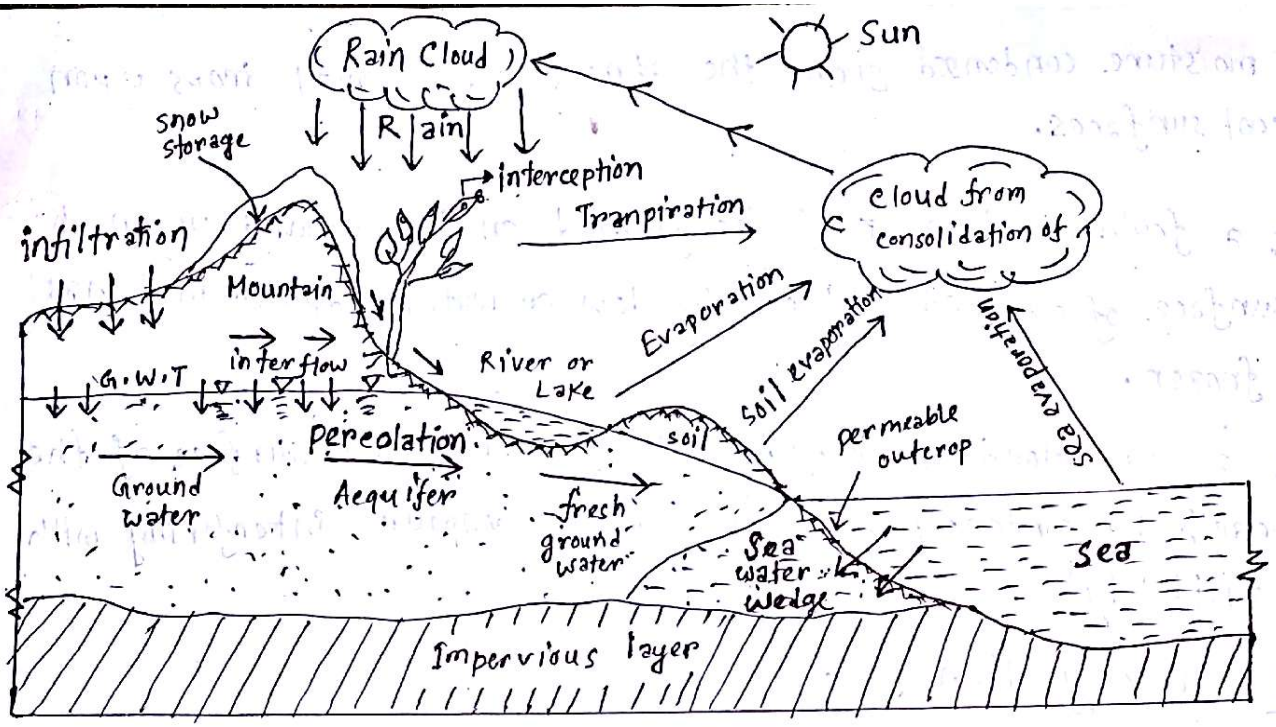

Fig. Hydrologic cycle.

Forms of precipitation:

1. Drizzle: a light steady in fine drops (0.5 mm) and intensity < 1 mm/hr.
2. Rain: The condensed water vapour of the atmosphere falling in drops (> 0.5 mm, maximum size - 6 mm) from the clouds.
3. Glaze: freezing of drizzle or rain when they come in contact with cold object.
4. Sleet: frozen rain drops while falling through air at subfreezing temperature.
5. Snow: ice crystal resulting from sublimation (i.e, water vapour condenses to ice)
6. Snow flakes: ice crystal fused together.
7. Hail: small lumps of ice (> 5 mm in dia.) formed by alternate freezing and melting, when they are carried up and down in highly turbulent air currents.

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8. Dew: moisture condensed from the atmosphere in small drops upon cool surfaces.

9. Frost: a feathery deposit of ice formed on the ground or on the surface of exposed objects by dew or water vapour that has frozen.

10. Fog: a thin cloud of varying size formed at the surface of the earth by condensation of atmospheric vapour (interfering with visibility)

11. Mist: a very thin fog.

Scope of Hydrology: 13, 12

The study of hydrology help us to know –

(i) the maximum probable flood that may occur at a given site and its frequency. This is required for the safe design of drains and culverts, dams and reservoirs, channels and other flood control structure.

(ii) the water yield from a basin – its occurence, quality and frequency etc; This is necessary for the design of dams, municipal water supply, water power, river navigation etc.

(iii) the ground water development for which a knowledge of the hydrology of the area; i.e. of the formation soil, recharge facilities like streams and reservoirs, rainfall pattern, climate, cropping pattern etc. are required.

(iv) the maximum intensity of storm and its frequency for the design of a drainage project.

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Importance of hydrology: 04

1. It gives an estimate of water resources potential of the river basin.

2. It provides an idea about probability of flood occurence their pattern and magnitudes.

3. The dependable yields for irrigation and hydro electric power stations can be determined.

4. spill way capacity, dam height, dam safety etc. can be designed.

5. Formulation of flood control measures can be carried out.

6. It helps in improvement of navigations, sediment carrying capacity etc.

7. It gives a guideline for erosion control.

8. Maximum expected flood discharge and its volume entering a reservoir can be determined.

Interception: Interception refers to precipitation that does not reach the soil, but is instead intercepted by the leaves, branches of plants and the forest floor.

Infiltration: Infiltration is the process by which water on the ground enters the soil and builds up ground water table.

percolation: Percolation is the process of the downward movement of water through unsaturated zone in to the phreatic ground water under the influence of gravity and hydraulic forces.

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Hydrological Data: 14

For the analysis and design of any hydrologic project adequate data and length of records are necessary. A hydrologist is often posed with lack of adequate data. The basic hydrological data required are:

(i) Climatological Data

(ii) Hydrometeorological data like temperature, wind velocity, humidity etc.

(iii) Precipitation records

(iv) stream flow records

(v) Seasonal fluctuation of ground water table or piezometric heads.

(vi) Evaporation data.

(vii) cropping pattern, crops and ground water

(viii) Water quality data of surface streams and ground water.

(ix) Geometric studies of the basin, like area, shape and slope of the basin, mean and median elevation, mean temperature and other physiographic characterestics of the basin; stream density and drainage density; tanks and reservoirs.

(x) Hydrometeorological characterestics of basin:

(a) a.a.r, long term precipitation, space average over the basin using isohyets and several other methods.

(b) Depth-Area-Duration (DAD) curves for critical storms.

(c) Isohyetal Maps

(d) cropping pattern

(e) Daily, monthly and annual evaporation from water surfaces in the basin.

(f) Water Balance study of the basin.

(g) chronic problems in the basin due to a flood-menacing river.

(h) Soil conservation and method of flood control.

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Hydrologic Equation: 02, 13, 14

The hydrologic equation is simply the statement of the law of conservation of matter is given by,

$$I = O + \Delta S$$

where, I = Inflow
O = Outflow
ΔS = change in storage.

This equation states that during a given period, the total inflow into a given area must equal the total out flow from the area plus the change in storage.

While solving this equation, the ground water is considered as an integral part of the surface water and it is the subsurface inflow and outflow that pose problems in the water balance studies of a basin.

Practical applications of Hydrology: 09

Some of the practical applications of Hydrology is:

(i) Design of hydraulic structure: The design of any structure related to water such as spillway, dam, culvert, bridge etc. largely depends on the knowledge of hydrology.

(ii) Municipal and Industrial water supply: The availability of water is often the most important factor in locating suitable place for industries and it has considerable effect on the growth of municipalities. Hydrology helps us to ensure water supply availability.

(iii) Irrigation: Storage of water is one of the most important factors for development of irrigation for which evaporation, seepage and other losses must be considered. Hydrology helps to measure these loses.

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(iv) Hydro power: Hydrologic studies are essential for the planning of any water power department.

(v) Flood control: The techniques of flood routine is essential for an economic planning of flood control project which can be known from the study of hydrology.

(vi) Navigation: To solve the hydrologic problems in navigation project some essential informations are required which is supplied by Hydrology.

(vii) Erosion and sediment control: The problems in erosion control in mainly related to overland flow and infiltration which are studied in hydrology.

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Precipitation

06

 # Precipitation: Precipitation is the product of the condensation of water vapors that fall under gravity.

From hydrological point of view, fall of moisture from atmosphere in any form on earth surface is called precipitation.

 # Formation of precipitation: 06

There are four essential requisities for the formation of precipitation:

(i) Accumulation of moisture: The simple principle of conservation of mass requires that a balance must be maintained between evaporation and precipitation. So, water is accumulated in atmosphere by evaporation and transpiration.

(ii) cooling: There are three types of cooling.

(a) Cyclonic cooling.
(b) Orographic cooling.
(c) convective cooling.

(iii) condensation: condensation takes place when the air is not saturated and contains small particles of substance that have an affinity to water These particles act as nuclei in the condensation process.

(iv) Growth of droplets: When the condensation nucleus attracts water, a droplet of water is produced and its size increases continuously. In about 24 hours, it becomes a drop of about 3mm. Its size further increases by collision with each other. When the droplets grows enough, it comes to the surface of earth as any form of precipitation.

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Types of Precipitation:

There are four types of precipitation. They are:

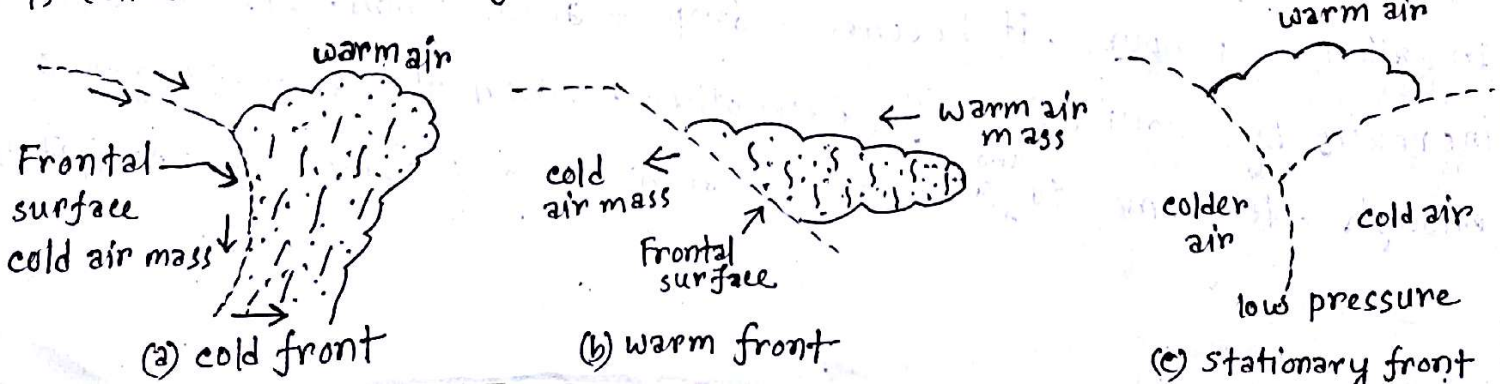
(i) Thermal convection (convectional precipitation): This type of precipitation is in the form of local whirling thunder storms and is typical of the tropics. The air close to the warm earth gets heated and rises due to its low density, cools adiabatically to form a cauliflower shape cloud, which finally bursts into a thunder storm. When accompanied by destructive winds, they are called 'tornados'.

(ii) conflict between two air masses (frontal precipitation): When two air masses due to contrasting temperatures and densities clash with each other, condensation and precipitation occur at the surface of contact. This surface of contact is called a 'front' or 'frontal surface'.

If a cold air mass drives out a warm air mass it is called a 'cold front'. And if a warm air mass replaces the retreating cold air mass, it is called 'warm front'.

On other hand, if the two air masses are drawn simultaneouly towards a low pressure area, the front developed is stationary and is called a 'stationary front'.

Cold fronts move faster than warm fronts and usually overtake them, the frontal surface of the cold and warm air sliding against each other. This phenomenon is called 'occlusion', and the resulting frontal surface is called an 'occluded front'.



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(iii) Orographic lifting (Orographic precipitation): The mechanical lifting of moist air over mountain barriers, causes heavy precipitation on the windward side.

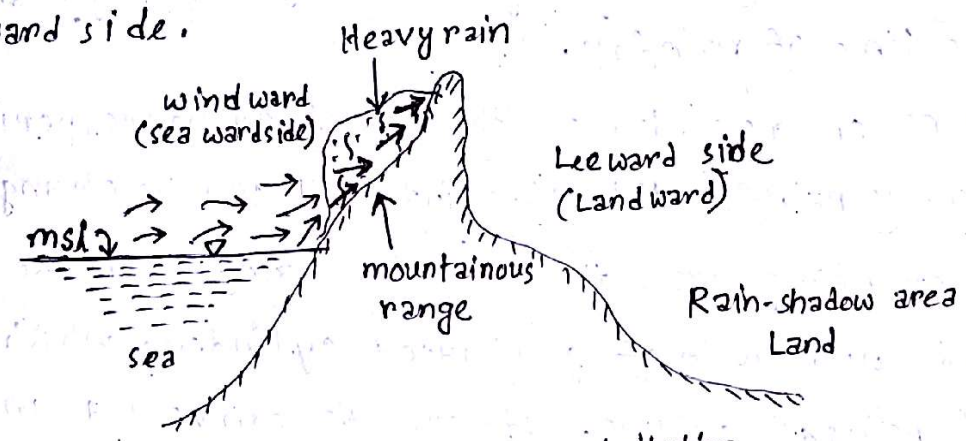

Fig. Orographic precipitation.

(iv) Cyclonic (cyclonic precipitation): This type of precipitation is due to lifting of moist air converging into a low pressure belt i.e, due to pressure differences created by the un equal heating of the earth surface. Here the winds blow spirally inward counter clockwise in the northern hemisphere and clockwise in the southern hemisphere. There are two main types of cyclones - (i) tropical cyclone / hurricane / typhoon. (ii) extra-tropical cyclone.

Measurement of precipitation:

Raingauge: Raingauge is an instrument, used for measuring the precipitation depth, falling over the land surface at a particular point and time.

Types of Raingauge: The raingauges are classified into two categories:

1. Non-recording type
2. Recording type

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1. Non recording type raingauge: This type of rain gauge records the rainfall depth occuring over the ground surface only, and unable to denote the time of rainfall.

The observations are recorded at the end of 24 hours period, but in case of heavy rainfall this time interval can be changed.

In this type of rain gauge, the collected rainfall in the 'can' is measured with help of graduated cylinder, which is obtained in terms of rainfall volume. It can be converted in the form of depth.

$$\text{Rain fall depth(cm)} = \frac{\text{volume of water collected } (cm^3)}{\text{Aperture area of the gauge } (cm^2)}$$

Symon's type raingauge and U.S weather Bureau raingauge have been introduced as non-recording type of raingauge.

Symon's type raingauge: This is one of the most popular and common type of non recording raingauge. It consists of a funnel with a circular rim of 12.7 cm diameter and a glass bottle as a receiver. The cylindrical metal casing is fixed vertically to the masonry foundation with level rim 30.5 cm above the ground surface. The rain falling into the funnel is collected in the receiver and is measured in a special measuring glass graduated in mm of rainfall, when it can measure 1.25 cm of rain.

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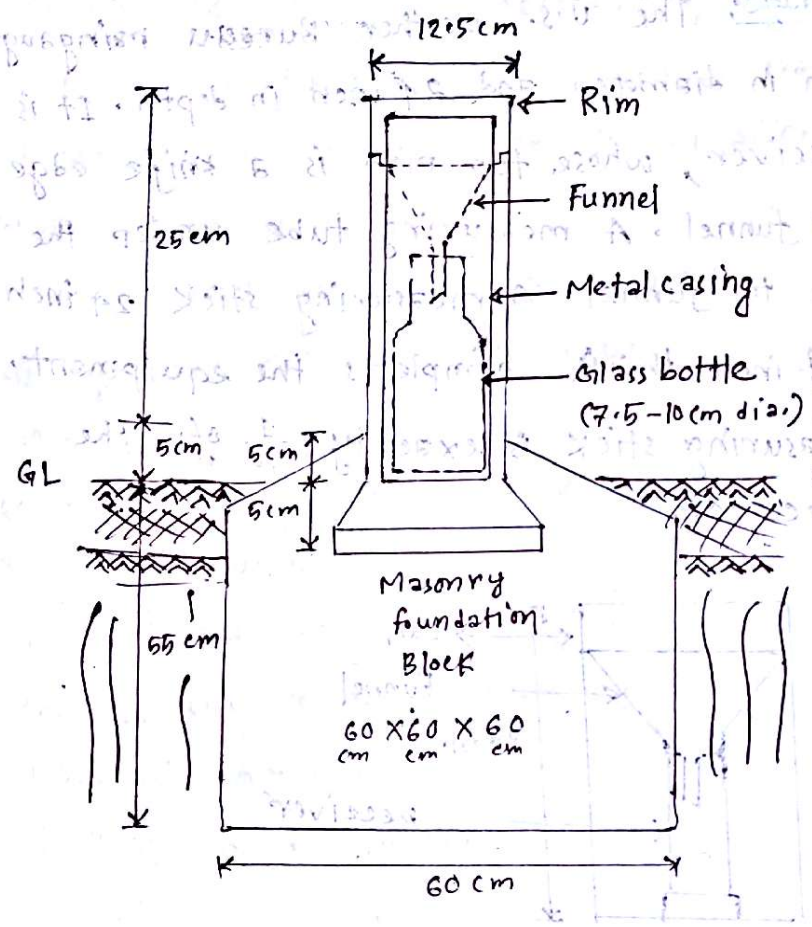

Fig. Symon's rain gauge.

Demerits of symon's raingauge:

(i) It does not give the intensity and duration of rainfall.

(ii) It does not provide the information on start and end of the rain fall.

(iii) The non recording type rain gauges are unable to give the accurates measurement, that can be compared with the rainfall amounts, measured by the other type of raingauges.

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U.S. Weather Bureau Raingauge: The U.S. weather Bureau raingauge consist of a 'can', 8 inch in diameter and 24 inch in depth. It is fitted over a copper receiver, whose top rim is a knife edge and bottom consist of a funnel. A measuring tube under the funnel takes water from the funnel. A measuring stick 24 inch long, $\frac{3}{8}$ inch wide and $\frac{1}{8}$ inch thick completes the equipment. The cross sectional of measuring stick is exactly $\frac{1}{10}$ of the opening area of the receiver.

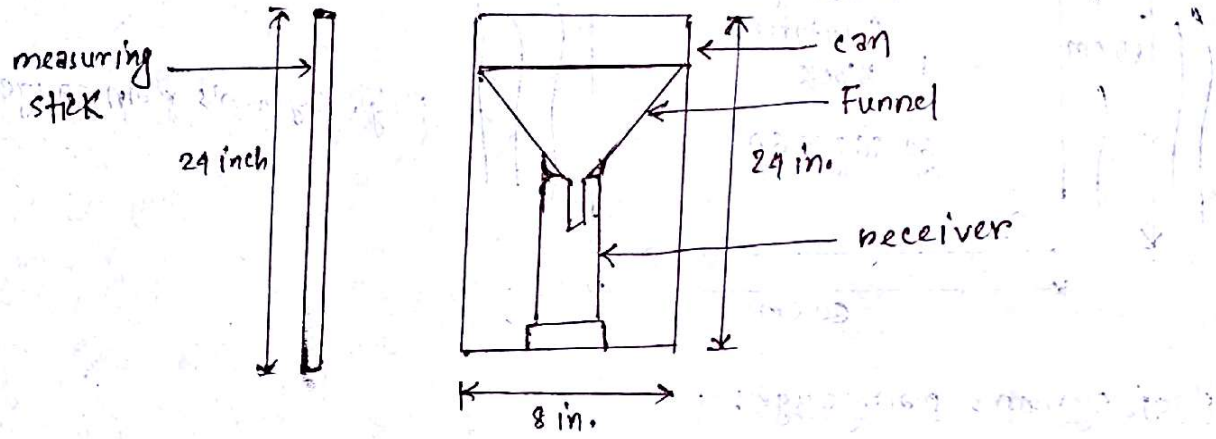

Fig. U.S. Weather Bureau raingauge

The rain water carried by the funnel is collected in the tube. The tube filled with water and containing measuring stick represents 2 inch rainfall. If the day's rainfall exceeds 2 inch, the excess water over flows in to the 8 inch can. In such cases, when the observer makes the reading, he first submerges the measuring stick, empties the measuring tube and pours excess water in to the tube for measurement.

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2. **Recording type raingauge:** Recording type raingauge are also called automatic, integrating or self recording raingauge; are those which record the rainfall amount and its duration simultaneously during storm occurance.

This type of raingauge has an automatic mechanical arrangement consisting of a clockwork, a drum with a graph paper fixed around it and a pencil point, which draws the mass curve of rain fall. Using the mass curve, the following informations can be determined:

(i) depth of the rainfall at any time
(ii) the intensity of rainfall at any instant during a storm.
(iii) time of onset of rainfall
(iv) cessation of rainfall.

Types of recording raingauge: There are three types of recording rain gauges:

(i) **Tipping bucket rain gauge:** This consists of a cylindrical receiver 30 cm diameter with a funnel inside. Just below the funnel a pair of tipping bucket is pivoted such that when one of the bucket receives a rainfall of 0.25 mm, it tips and empties into a tank below, while the other bucket takes its position and the process is repeated.

Limitations:
(i) It takes very little time to tip, therefore some amount of rainwater is over flowed from the bucket, hence measurement is not absolutely correct.
(ii) It is difficult to characterize the time of start and end of the rainfall.

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(iii) In light shower, the rain gauge becomes useless.

(iv) Due to sophistication in its mechanism, it requires a constant servicing after a fixed time period use.

(ii) Weighing type rain gauge: In this type of rain gauge, when a certain weight of rainfall is collected in a tank, which rests on a spring-lever balance, it makes a pen to move on a chart wrapped round a clock driven drum. The rotation of the drum sets the time scale while the vertical motion of the pen records the cumulative precipitation.

(iii) Float type rain gauge: It is also known as syphon type raingauge. In this type, as the rain is collected in a float chamber, the float moves up which makes a pen to move on a chart wrapped round a clock driven drum. When the float chamber fills up, the water siphons out automatically through siphon tube kept in an interconnected siphon chamber. The clockwork revolves the drum once in 24 hours. The clock mechanism needs rewinding once in a week when the chart wrapped round is also replaced.

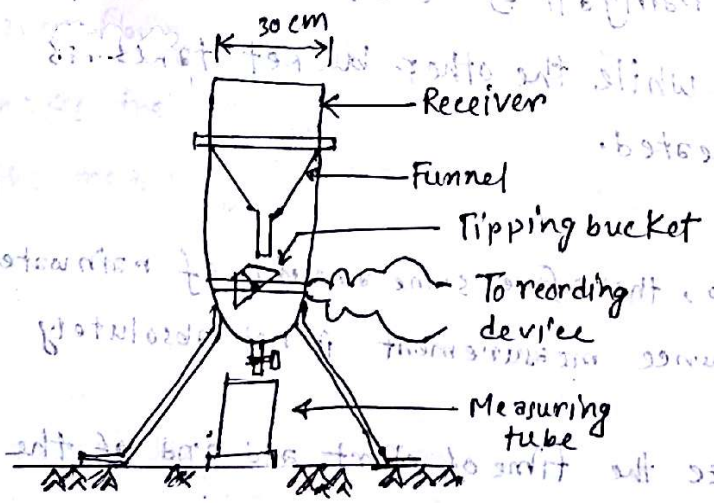

Fig. Tipping Bucket Gauge

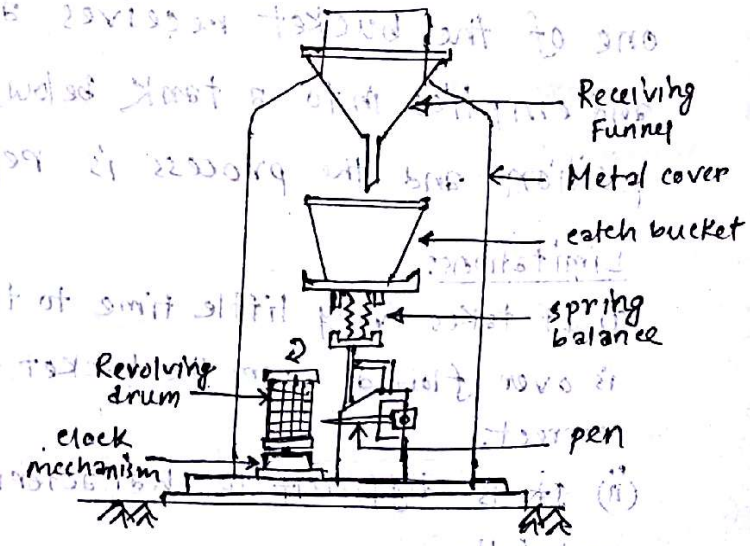

Fig. Weighing type raingauge.

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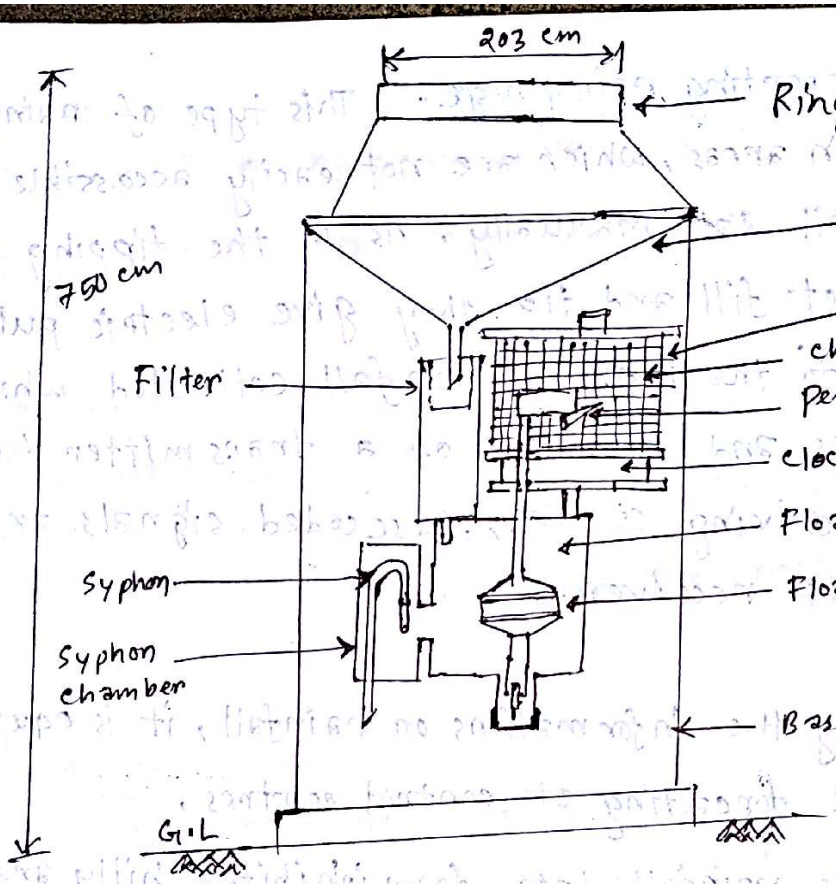

Fig. Float type rain gauge

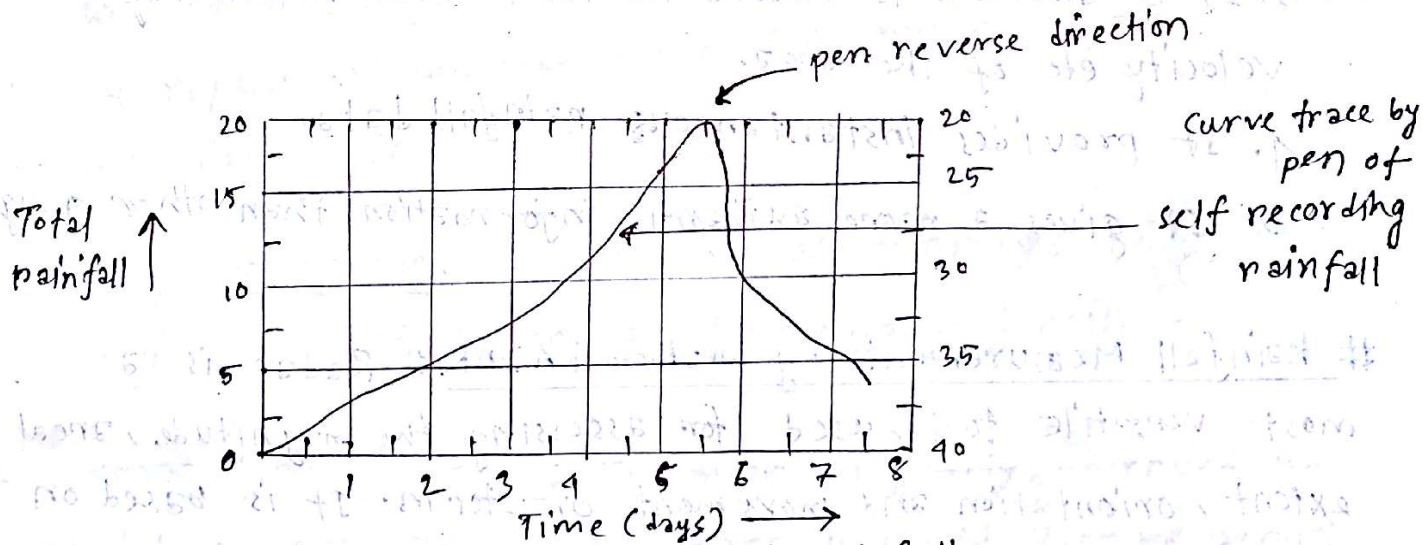

Fig. Mass curve of rainfall

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Automatic radio-reporting rain gauge: This type of rain gauge is used in mountain areas, which are not easily accessible to collect the rainfall data manually. As in the tipping bucket gauge, when bucket fill and tip, they give electric pulses equal in number to the mm of rainfall collected which are coded in to mesages and impressed on a transmitter during broadcast. At receiving station, these coded signals are picked up by UHF receiver.

Advantages:

1. Besides providing the informations on rainfall, it is equally applied for flood forcasting at control centres.
2. It supplies the rainfall data from inhibited hilly areas.
3. It is also able to record the temperature, humidity, wind velocity etc of the area.
4. It provides instantaneous rainfall data.
5. It gives a more authentic information than other gauge.

Rainfall Measurement by weather RADAR: Radar is a most versatile tool, used for assessing the magnitude, areal extent, orientation and movement of storm. It is based on the principle of echo-sounding. For rainfall measurement high electromagnetic signals are released with the speed equal to the speed of light in the sky, some portion of which is absorbed by the rain, is detected by the radar.

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Advantages:

1. It gives 24 hr rainfall intensity.
2. It is adopted for large area and inaccessible catchment areas.
3. It provides instantaneous precipitation map for a vast watershed area.
4. It gives space as well as time wise continuous measurement.
5. The data obtained by radars are more precise than other raingauges.

Disadvantages:

1. It is very-very costly set up.
2. Like raingauge, a network of several radars can not be installed.
3. Well trained engineering staffs are required to operate it.
4. Its repair and maintenance cost is much more higher than other conventional instruments.

09, 13, 14

Rain gauge density: The number of raingauges to be erected in a given area is termed as 'Rain gauge density'.

08, 14, 15

Average annual rainfall: The mean of yearly rainfall observed for a period of 35 consecutive years is called average annual rainfall (a.a.r)

The a.a.r of a place depends upon: (i) distance from ocean
(ii) direction of prevailing winds
(iii) the mean annual temperature
(iv) altitude of the place and
(v) its topography.

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Index of wetness: The ratio of rainfall in a particular year to the a.a.r is called 'the index of wetness'. For an example, an index of wetness 60% in a particular year indicates a rainfall deficiency of 40%.

Wet Year: The year in which the rainfall is greater than a.a.r, is called wet (good) year.

Dry year: The year in which the rainfall is less than a.a.r is called dry (bad) year.

Normal year: The year in which the rainfall is equal to a.a.r is called normal (average) year.

Isohyet: A imaginary line joing the places having the same precipitation or a.a.r is called an 'isohyet'.

Arid climate: If the a.a.r < 40 cm, then it is called arid climate. In an arid climate, a drought is a normal state of affairs.

Semi-Arid climate: If the a.a.r is 40 to 75 cm, then it is called semi-arid climate. In semi-arid climate, a drought occurs at least once a year except in abnormal years.

Humid climate: If the a.a.r > 75 cm., then it is called humid climate. In a humid climate, a drought does not occur in ordinary years.

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Advantages and Disadvantages of Recording type raingauge:

Advantages: 16

1. It provides a detail information about the rainfall such as rainfall depth and its duration.
2. It automatically records the rainfall amount with respect to time.
3. There is no need to engage any attendant for recording.
4. The introduction of error in the rainfall data by man mistakes is completely eliminated.
5. It is the only means to record rainfall data in hilly areas.
6. It provides the information about onset and cessation of rainfall.
7. It denotes the entire rainfall information in the form of graph, which is used as permanant tool for making the analysis of storm.
8. Measuring capacity of recording type rain gauge is more to that of non-recording type rain gauge.
9. Duration of rainfall is also recorded.
10. It provides more authentic result than non-recording type rain gauge.

Disadvantages:

1. Its cost is more than the non recording type gauge.
2. It requires a regular service for its proper working.
3. Its maintenance cost is comparatively high.

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4. Additional precautions are required to operate this gauge.

5. Recording of the rain fall data may be interrupted due to some mechanical faults incurred in the gauge.

07, 16

Differences between Recording and Non-recording type rain gauge:

Recording Raingauge	Non-recording raingauge
1. This type of raingauge records rain fall amount and its duration simultaneously.	1. This type of raingauge records rainfall depth only.
2. Initial cost is more than non-recording type raingauge.	2. Initial cost is comperatively less.
3. Measuring capacity is more	3. Measuring capacity is less
4. Error by man mistake is completely eliminated.	4. There is a possibility of error by man-mistake
5. Maintenance cost is comperatively high.	5. Maintenance cost is comparatively low.
6. Rainfall in hilly area can be measured accurately.	6. Rainfall data in hilly areas can not be measured
7. Recording of rainfall data may be interrupted by mechanical faults.	7. There is no possibility of mechanical faults.
Example: Symon's type raingauge, U.S weather bureau raingauge.	Example: weighing type, Tipping bucket, Float type raingauge

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06, 08, 10, 11, 12, 13, 15, 16

Installation of Raingauges: The following points ~~such~~ should be kept in view for installing the raingauge:

1. The ground surface must be level.
2. Generally the locations such as sloping ground, terrace wall or roof are avoided for locating the raingauge.
3. The site should be such that, it can represent the entire water shed.
4. The wind affected areas should be avoided for locating the raingauge.
5. The site should be open.
6. The raingauge must be shield by fencing atleast 5.5 x 5.5 m^2 area around, in hilly areas.
7. The ideal location of raingauge in the forest area needs that, it should be fixed at a distance of twice the height of tallest tree from the forest
8. Location of raingauge should be neglected in the valley or at ridge point of the area.
9. The surface, over which raingauge is being installed such that, the receiver's height is 75 cm above the ground surface
10. Raingauge should be installed in vertical position.
11. The installation of raingauge should be preferred in easily accessible area.

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common errors of Rainfall measurements:

1. The data obtained from the measurement of rainfall using non-recording type raingauge, may be erroneous due to displacement of water level by measuring scale.

2. Atmospheric temperature also yields some error by increasing the evaporation loss.

3. Splashing of water from the collecting funnel, yields some error.

4. Existing of bends and dents in the collector, makes inaccurate measurement of rainfall.

5. In-accurate recording of the observation, produces errors in the measurement,

6. Frictional effect in recording mechanism introduces erroneous effect.

7. The obstructions like tree, building etc. creat the errors by affecting the wind pattern of the location,

8. Some volume of water is lost in wetting of initially dried surface of the measuring gauge.

These are responsible for introducing the error in rainfall measure.

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Determination of optimum number of raingauge: 01, 03, 05, 09, 10

The procedure for determining the optimum number of raingauge to be installed in a given basin is explained below:

(i) Mean annual rainfall is calculated from the data recorded at existing raingauge station by using the relation:

$$\bar{P} = \frac{\sum_{i=1}^{n} P_i}{n} = \frac{P_1 + P_2 + \cdots + P_n}{n}$$

where, $\bar{P}$ = mean annual rainfall

n = total number of raingauge

$P_1, P_2, \cdots, P_n$ = Annual rainfall recorded in station 1, 2 ... etc.

(ii) The standard deviation, σ is determined from existing station's record using the relation,

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n} (P_i - \bar{P})^2}{n-1}}$$

(iii) co-efficient of variation, C_v is calculated as

$$C_v = \frac{\sigma}{\bar{P}} \times 100$$

(iv) Optimum number of raingauge required to install is calculated from the following relation:

$$N = \left(\frac{C_v}{\varepsilon}\right)^2$$

where, C_v = co-efficient of variation of the rainfall of the existing raingauge station.

ε = Percentage allowable error

N = optimum number of raingauge required

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Estimates of missing data and Adjustment of records: 2011

For frequency analysis of rainfall data, a sufficiently long record is required. It may so happen that a particular rain gauge is not operative for part of a month or so (since it is broken or for some other reason), when it becomes necessary to supplement the missing record by one of the following methods:

1. Arithmatic Mean Method:

For use of this method, the following conditions must be satisfied:

(i) There should be available the rainfall records of three stations, at least.

(ii) All the stations should be equally spaced around the station of which, data is missed.

(iii) Normal annual rainfall at the missing station should be with in 10% of the normal annual rainfall at the surrounding the index stations.

Missing rainfall amount (P_x) at station x can be determined by averaging the annual precipitation values of the index stations:

$$P_x = \frac{1}{n}\left(P_1 + P_2 + P_3 + \cdots + P_n\right)$$

2. Normal Ratio Method: 2015, 2016

The method is used when,

(i) Normal annual precipitation at index stations differs by more than 10% from the stations, of which rainfall data is missed.

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(ii) At least three station's rainfall data is available.

(iii) All the stations are evenly spaced.

Let, $N_a, N_b, N_c \ldots\ldots, N_n$ are the normal annual precipitation at index stations A, B, C ....., N respectively. $P_a, P_b, P_c \sim P_n$ are the rainfall recorded at those stations, respectively.

if N_x is the normal annual precipitation of station x of which data is missed, then the missing rainfall amount (P_x) can be determined:

$$P_x = \frac{1}{n}\left[\frac{N_x}{N_a} \cdot P_a + \frac{N_x}{N_b} \cdot P_b + \frac{N_x}{N_c} \cdot P_c + \ldots\ldots + \frac{N_x}{N_n} \cdot P_n\right]$$

3. Graphical Method:

The prediction of missing rainfall data is based on only two index station. In this method, The rainfall data of both stations are plotted on a graph paper, keeping one station's record on ordinate and other station's reord on abassica, and a straight line is drawn by passing approximately through middle of all the plotted points, on own adjudgement. With the help of straight line obtained, the missing rainfall value is predicted corresponding to known rainfall amount of index station.

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4. National weather service method: (Weighted Average of four Stations)

In this method, Four quadrants are delineated by north-south and east west lines passing through the raingauge station where the missing rainfall is to be estimated. One raingauge station in each quadrant, which is the nearest to the raingauge station under question in that quadrant, is selected. If P_1, P_2, P_3, P_4 are the rainfall recorded at the raingauge station in the four quadrants which are nearest to station 'X' in the respective quadrants, and which are at distances of r_1, r_2, r_3 and r_4 from station 'X' respectively, Then the missing rainfall amount at station 'x' is;

$$P_x = \frac{\left[\frac{P_1}{r_1^2} + \frac{P_2}{r_2^2} + \frac{P_3}{r_3^2} + \frac{P_4}{r_4^2}\right]}{\left[\frac{1}{r_1^2} + \frac{1}{r_2^2} + \frac{1}{r_3^2} + \frac{1}{r_4^2}\right]}$$

5. Long Term mean rainfall for a New station Method:

This method assumes that, Both adjacent stations are identical and represent similar hydrological condition.

The long term rainfall amount of station B can be determined by:

$$\bar{B}_{lt} = \frac{\bar{A}_{lt} \cdot \bar{B}_{st}}{\bar{A}_{st}}$$

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Mean Areal Depth of precipitation: 2015

Point Rainfall – It is the rainfall at a single station. For small areas less than 50 km^2, point rainfall may be taken as the average depth over the area.

But, as the rainfall over a large area is not uniform, the the average depth of rainfall over the area can be determined by the following three methods:

(i) Arithmatic average method: It is obtained by simply averaging arithmetically the amount of rainfall at the individual raingauge stations in the area. i.e.

$$P_{ave} = \frac{\sum_{i=1}^{n} P_i}{n}$$

where, P_{ave} = average depth of rainfall over the area

ΣP_i = sum of rainfall amounts at individual raingauge stations.

n = number of raingauge stations in the area.

(ii) Thiessen polygon method: In this method, the stations are plotted in a map and joined by straight lines, perpendicular bisectors are drawn to the straight lines, joing adjacent stations to form polygons. Means areal depth is determined as —

$$P_{ave} = \frac{\sum_{i=1}^{n} A_i P_i}{\sum_{i=1}^{n} A_i}$$

where, $P_i = P_1, P_2 \ldots$ = The rainfalls at the individual stations

$A_i = A_1, A_2 \ldots$ = Areas of the polygons surrounding these stations.

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(iii) The Isohyetal Method: In this method, the point rainfalls are plotted on a suitable base map and the lines of equal rainfall (isohyets) are drawn giving consideration to orographic effect and storm morphology. The average depth can be determined as;

$$P_{ave} = \frac{\sum_{i=1}^{n} A_{i-(i+1)} P_{i-(i+1)}}{\sum_{i=1}^{n} A_{i-(i+1)}}$$

where, $A_{i-(i+1)}$ = area between two successive isohyets P_i and P_{i+1}

$$P_{i-(i+1)} = \frac{P_i + P_{i+1}}{2}$$

$$\Sigma A_{i-(i+1)} = \text{Total area of the basin.}$$

Saturated Network Design:

If the project is very important, the rainfall has to be estimated with great accuracy; then a network of rain-gauge stations should be so set up that any addition of rain-gauge stations will not appreciably alter the average depth of rainfall estimated. Such a network is referred to as a saturated network.

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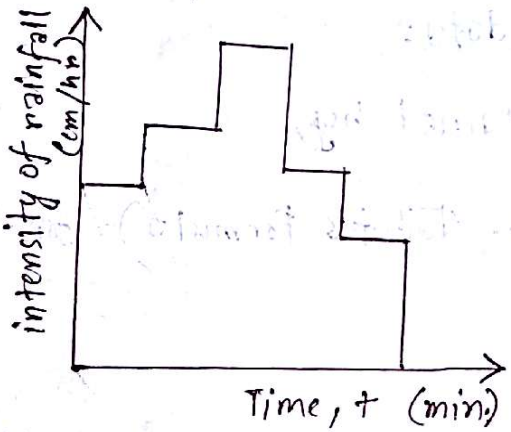
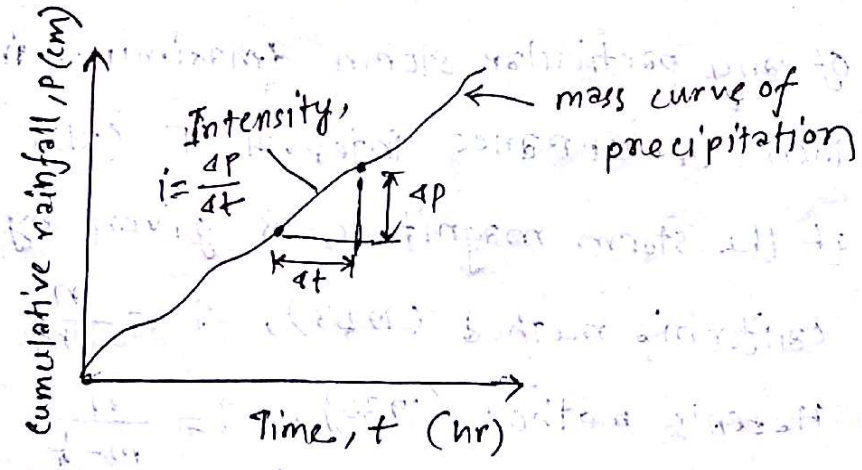
Graphical Representation of Rainfall:

The variation of rainfall with respect to time may be shown graphically by (i) a hyetograph (ii) a mass curve.

Hyetograph: A hyetograph is a bar graph showing the intensity of rainfall with respect to time. It is useful in determining the maximum intensities of rainfall during a particular storm as is required in land drainage and design of culverts.



Mass Curve: A mass curve of rainfall is a plot of cumulative depth of rainfall against time. From the mass curve, the total depth of rainfall and intensity of rainfall at any instant of time can be found.



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Characterestics of Rainstorm:

The characterestics of Rainstorm are:

(i) Intensity (cm/hr)

(ii) Duration (mins, hr, or days)

(iii) frequency (once in 5 yrs or once in 10, 20, 40, 60 or 100 yrs)

(iv) Areal extent (i.e. area over which it is disturbed)

Analysis of rainfall data:

Intensity can be obtained by,

(i) $i = \frac{a}{t+b}$ (A.N. Talbot's formula) for (t = 5–120 min.)

(ii) $i = \frac{K}{t^n}$

(iii) $i = Kt^x$ where, t = duration of rainfall

a, b, K and x are constants for a given region.

If there are a total number of n items and the order number or rank of any particular storm (maximum intensity or depth) is m, then recurrence interval T (also known as return period) of the storm magnitude is given by:

(a) California method (1923), $T = \frac{n}{m}$

(b) Hazen's method (1930), $T = \frac{n}{m-\frac{1}{2}}$

(c) Kimball's method (Weibull, 1939), $T = \frac{n+1}{m}$

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And the frequency F of the storm magnitude is given by-

$$F = \frac{1}{T} \times 100\%$$

The probability that a T-year storm (and frequency, F) may not occur in any series of N years is,

$$P(N,0) = (1-F)^N$$

and that it may occur is

$$P_{Ex} = 1-(1-F)^N$$

where, P_{Ex} = probability of occurance of T year storm in N years,

If the intensity duration curves are plotted for various storm, for different recurrence interrals, then a relation may be obtained of the form:

$$i = \frac{KT^x}{t^e} \quad \text{(Sherman)}$$

where, h, x and e are constant.

Moving Average Curve:

In order to depict a general trend in the rainfall pattern, the averages of three or five consecutive years are found out progressively by moving group average, one year at a time.

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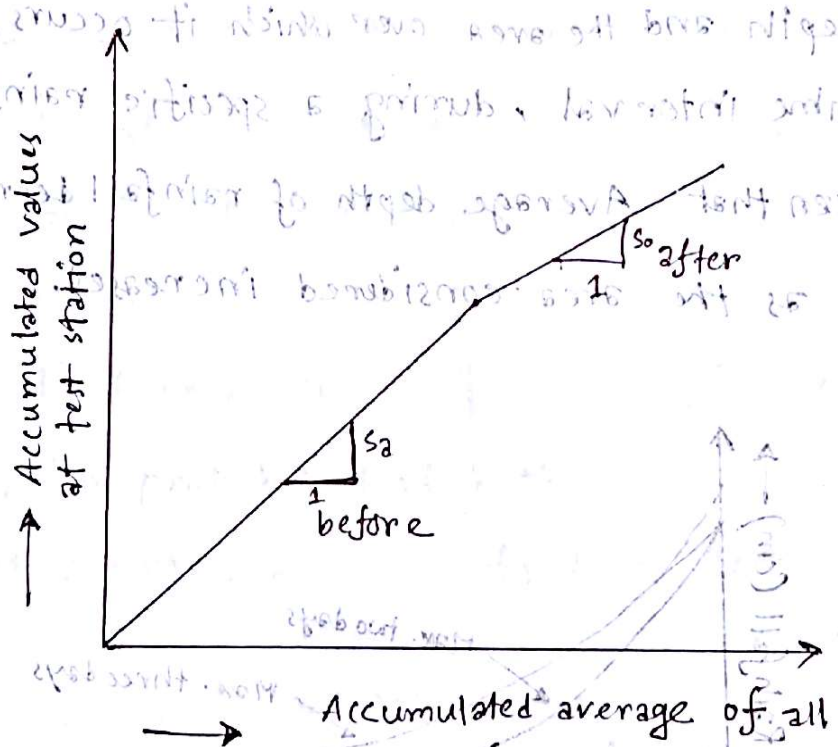
Double Mass Curve:

Double Mass curve is a curve which is used to check the consistency of many kinds of hydrologic data by comaparing data for a single station with that of a pattern composed of the data from several other stations in the area.

It can be used to adjust inconsistance precipitation data.



Adjusted Precipitation, $P_a = \left(\frac{S_a}{S_o}\right) \times P_o$ → observed precipitation.

S_a ↑ slope prior to break

S_o ↓ slope after the break

Water Losses

Various water losses: 2009, 06

(i) Interception loss: due to surface vegetation

(ii) Evaporation:
   (a) from water surface i.e. lakes, ponds, rivers etc.
   (b) from soil surface

(iii) Transpiration: from plant leaves

(iv) Evapotranspiration or consumptive use: from irrigated or cropped land.

(v) Infiltration: into the soil at the ground surface

(vi) Watershed leakage: ground water movement from one basin to another or into the sea.

Interception Loss:

The precipitation intercepted by foliage (plant leaves, forests) and its buildings and returned to atmosphere (by evaporation from plant leaves) without reaching the ground surface is called interception loss.

Interception loss is high in the beginning of storms and gradually decreases.

Effective Rain = Rainfall - Interception loss

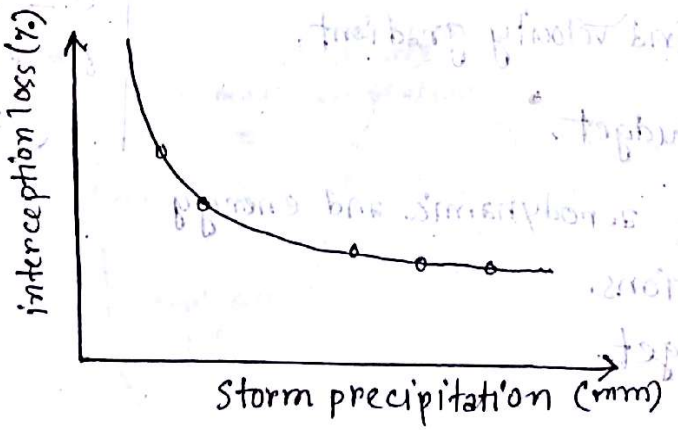


Fig. Horton's mean curve of Interception loss

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Factors affecting Evaporation:

The factors affecting evaporation are:

(i) Air.
(ii) Water temperature.
(iii) Relative humidity.
(iv) Wind velocity.
(v) Surface area.
(vi) Barometric pressure.
(vii) Salinity of the water.

(The last two having a minor effect.)

Methods of Estimating Lake Evaporation:

Evaporation from water surfaces can be determined from the following methods:

(i) The storage equation

$$P + I \pm O_g = E + O \pm S$$

where, P = precipitation
I = surface inflow
O_g = subsurface inflow or outflow
E = Evaporation
O = surface out flow
S = change in surface water storage.

(ii) Auxiliary pans.
(iii) Evaporation formula.
(iv) Humidity and wind velocity gradient.
(v) The energy Budget.
(vi) combination of aerodynamic and energy balance equations.
(vii) The water Budget.

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Pan Co-efficient: 2015, 2014, 07

Evaporation pan data can not be applied to free water surfaces directly but must be adjusted for the distances in physical and climatological factors.

The small volume of water in the metallic pan is greatly affected by temperature fluctuations in the air in contrast with large bodies of water with little temperture fluctuations.

Thus the pan evaporation data have to be corrected to obtain the actual evaporation from water surfaces of lakes and reservoirs, i.e. by multiplying by a co-efficient called pan co-efficient.

$$\text{Pan co-efficient} = \frac{\text{Lake evaporation}}{\text{Pan evaporation}}$$

And the experimental values of pan co-efficient range from 0.67 to 0.82 with an average of 0.7.

Measures to reduce Lake Evaporation: 2014, 13, 10, 08

The following are some of the recommended measures to reduce evaporation from water surface:

(1) Storage reservoirs of more depth and less surface area.

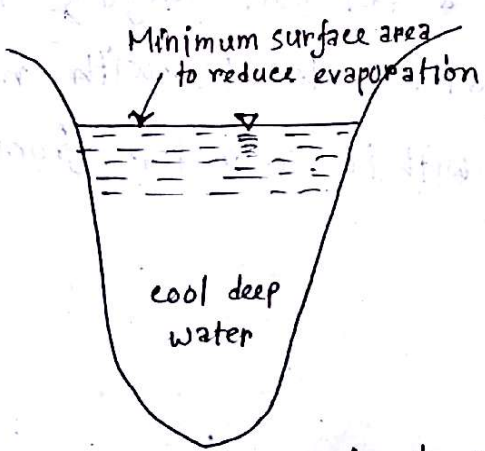

Fig. Reservoir in a deep gorge

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2. By growing Tall trees like casuerina on the windward side of the reservoir.

3. By spraying certain chemicals or fatty acids and formation of film.

4. By allowing flow of water, temperature is reduced and evaporation is reduced.

5. By removing the water loving weeds and plants like phreatophytes from the peripheri of the reservoir.

6. By straightening the stream channels, the exposed area of the water surface is reduced and hence evaporation is reduced.

7. By providing mechanical coverings like thin polythene sheet to small agricultural ponds and lakes.

8. By developing underground reservoir.

9. If the reservoir is sorrounded by huge trees and forest, the evaporation will be less due to cooller environment.

Soil Evaporation:

The evaporation from a wet soil surface immediately after rain or escape of water mulecules with more resistance when water table lies within a meter from ground, is called soil evaporation.

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This expressed as a percentage of evaporation from free water surface is called evaporation oppartunity.

$$\text{Evaporation opportunity} = \frac{\text{Actual Evaporation from the land at a given time}}{\text{Evaporation from an equivalent water surface}} \times 100$$

Transpiration:

Transpiration is the process by which the water vapour escapes from the living plant leaves and enters the atmosphere.

Transpiration Ratio:

Transpiration ratio is the ratio of the weight of water absorbed, conveyed through and transpired from a plant during growing season to the weight of dry matter produced exclusive of roots.

Evapotranspiration:

Evapotranspiration or Consumptive use is the total water lost from a cropped land due to evaporation from the soil and transpiration by the plants or used by the plants in building up of plant tissue.

Potential Evapotranspiration:

Potential Evapotranspiration is the Evapotranspiration from the short green vegetation when the roots are supplied with unlimited water covering the soil.
It is usually expressed as a depth (cm, mm) over the area.

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Estimation of Evapotranspiration:

The following are the methods of estimating evapotranspiration.

(i) Tanks and lysimeter experiments.

(ii) Field experimental plots.

(iii) Installation of sunken tanks.

(iv) Evapotranspiration equations.

(v) Evaporation index method.

Factors affecting Evapotranspiration:

The following factors affect Evapotranspiration:

(i) Climatological factors like:

- (a) percentage sunshine hours
- (b) wind speed
- (c) mean monthy temperature
- (d) Humidity.

(ii) Crops factors like:

- (a) Type of crop
- (b) the percentage growing season.

(iii) The moisture level in the soil.

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Infiltration:

Water entering the soil at ground surface is called Infiltration.

Infiltration capacity:

The maximum rate at which the soil in any given condition is capable of absorbing water is called its infiltration capacity.

Infitration Curve:

Infiltration often begins at a high rate and decreases to a fairly steady rate as the rain continues, called ultimate infiltration. (f_p).

The infiltration rate(f) at any time t is given by Horton's equation:

$$f = f_c + (f_o - f_c)e^{-Kt}$$

Where, $K = \frac{f_o - f_c}{F_c}$

Here,

f_o = initial rate of infiltration capacity

f_c = final constant rate of infiltration at saturation

K = a constant depending primarily upon soil and vegetation

e = base of the Napierian logarithm

F_c = shaded Area in Figure.

t = time beginning of the storm.

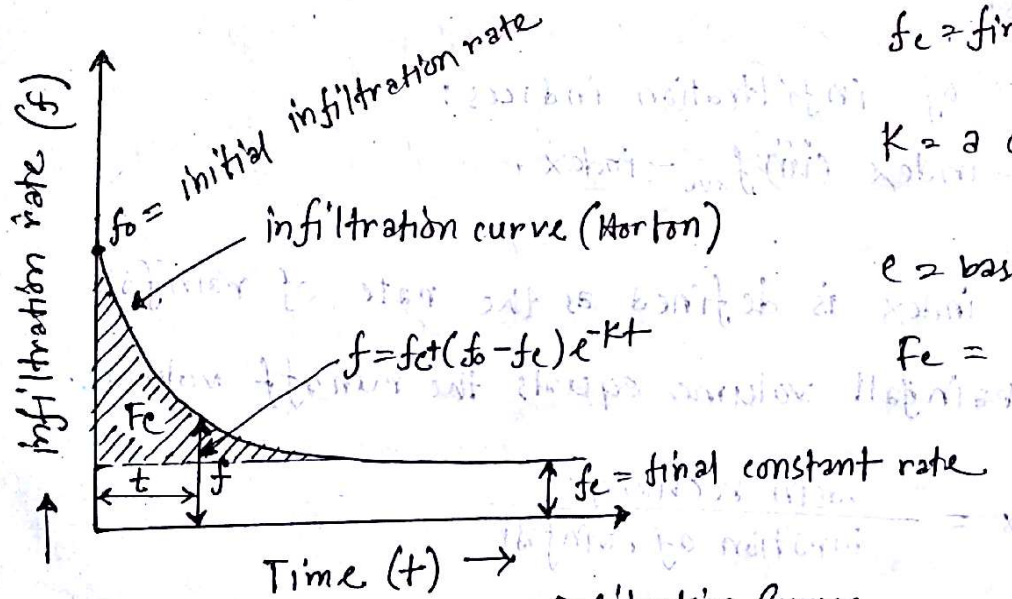

Fig. Infiltration Curve

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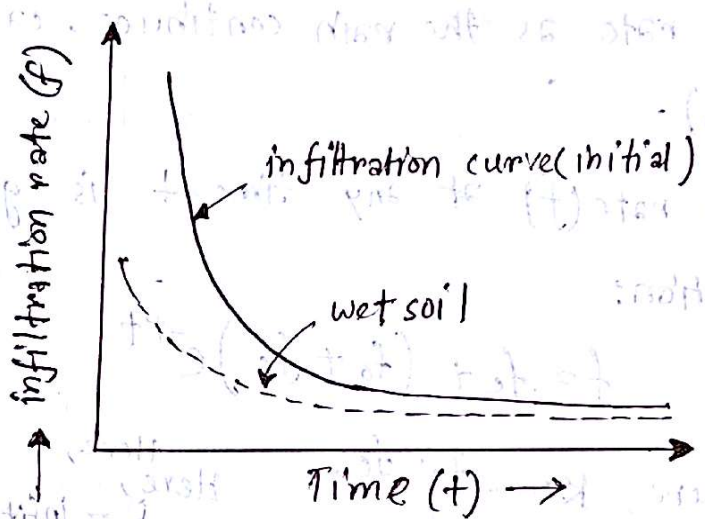
Methods of Determining Infiltration:

The methods of determining infiltration are:

(i) Infiltrometers
(ii) Observation in pits and ponds
(iii) Placing a catch basin below a laboratory sample
(iv) Artificial rain simulator.
(v) Hydrograph analysis.

Typical Infiltration Curve: 00, 02, 05, 06, 07



Infiltration indices:

There are three types of infiltration indices:

(i) ϕ-index (ii) W-index (iii) f_{ave}-index.

ϕ-index: The ϕ-index is defined as the rate of rainfall above which the rainfall volume equals the runoff volume.

$$\phi\text{-index} = \frac{\text{basin recharge}}{\text{duration of rainfall}}$$

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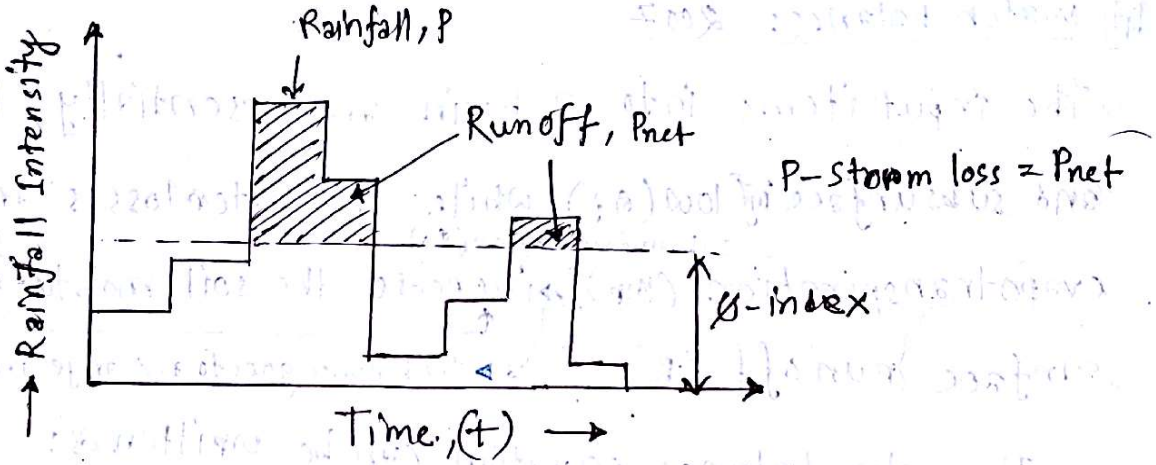

Fig. Infiltration loss by ϕ-index

W-index: The w-index is the average infiltration rate during the time rainfall intensity exceeds the infiltration capacity rate, i.e.

$$W\text{-index} = \frac{f_p}{t_R} = \frac{P-Q-S}{t_R}$$

where,

P = Total Rainfall
Q = Surface runoff
S = Effective surface retention.
t_R = Duration of storm during which $i > f_p$
F_p = Total infiltration.

f_{ave}-index: In this method, an average infiltration loss is assumed throughout the storm, for the period, $i > f$

Watershed Leakage: 2007

The rain after infiltering into the ground may percolate through the subsoil and build up the ground water table (GWT). This ground water through the water bearing strata may flow into the adjacent basin or directly into the sea. If the water bearing strata outcrops into the sea. This is called Watershade Leakage.

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Water Balance: 2007

The input items into a basin are essentially precipitation (P) and subsurface inflow (G_i) while the water losses are evaporation (E), subsurface outflow (G_o). evapotranspiration (E_t), increase the soil moisture (SMA) and as surface runoff (R). The balance goes to recharge ground water (G_r)

The water balance equation can be written as:

$$P + G_i = E + E_t + G_o + SMA + G_r + R$$

2012, 2015, 06

Double Ring Infitrometer method of determining infiltration:

(i) A double ring infiltrometer is penetrated into the soil to a depth of 15 cm ~~with a hammer~~ by a driving plate and hammer. with out any tilt or undue disturbance of soil surface.

(ii) After driving is over, any disturbed soil adjacent to the sides tamped with a metal tamper.

(iii) point gauges are fixed in the center of the rings and in the annular space between the two ring.

(iv) water is poured into the rings to maintain the desired depth

(v) To maintain constant depth, water is added continuously at regular time intervals up to the period of at least 6 hours.

(vi) The results are plotted as infiltration rate (cm/hr) versus time (min.)

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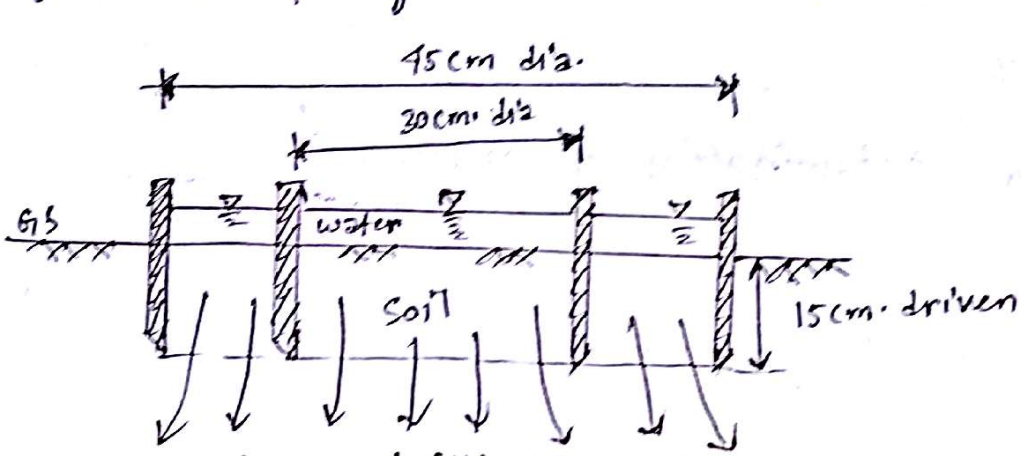
Infiltration capacity is determined from the curve.



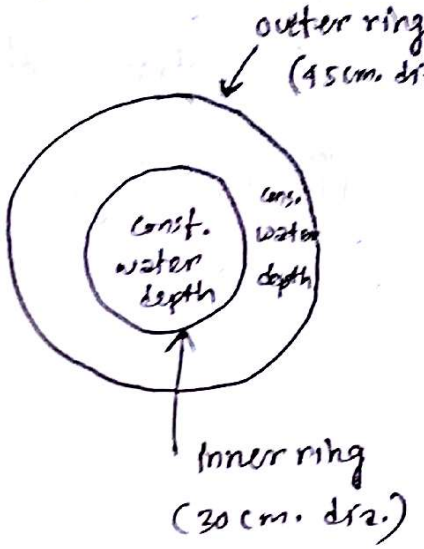


Fig. Double ring infiltrometer.

Factors affecting Infiltration: 2014, 11, 08, 07, 06, 04, 02, 01

(1) Rainfall factors: (i) Rainfall intensity
(ii) Rainfall Duration
(iii) Forms of precipitation

(2) Soil Factors: (i) Soil Type.
(ii) Soil surface slope,
(iii) soil moisture.

(3) Surface cover factor: (i) vegetation type
(ii) Agricultural crops
(iii) Snow cover.

(4) Climate factor: (i) Atmospheric temperature.
(ii) Soil Temperature.
(iii) Seasonal variation.

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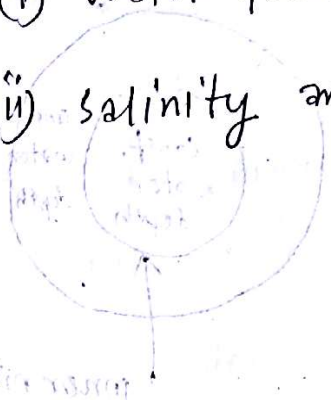
Other factors:

(i) water quality

(ii) salinity and contamination of water.



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RUNOFF

 # Runoff: The portion of precipitation which appears in the surface streams of either perennial or intermittent nature is called Run-off.

Total run off may be expressed as -

Runoff = surface runoff + sub surface runoff + Ground water run off or Base flow.

 # Types of Run-off:

Run off is broadly classified into following three types:

(i) surface run-off (ii) Subsurface runoff (iii) Base flow

(i) **surface run-off:** The portion of rainfall which enters the streams, channels etc. immediately after occuring the rainfall is called surface run off. It flows as a thin sheet of water over the land surface.

(ii) **subsurface run-off:** The amount of rainfall which first leaches in to the soil and then starts flowing laterally without joining watertable to the streams, rivers etc. is called sub surface runoff.

(iii) **Base flow:** the part of rainfall which after falling over the ground surface, percolates in to the soil and meets watertable and finally joins to the streams, oceans etc. is called Base flow.

 # Depression storage: When the overland flow or surface flow starts due to a storm some flowing water is held in puddles, pits and small ponds; this water stored is called depression storage.

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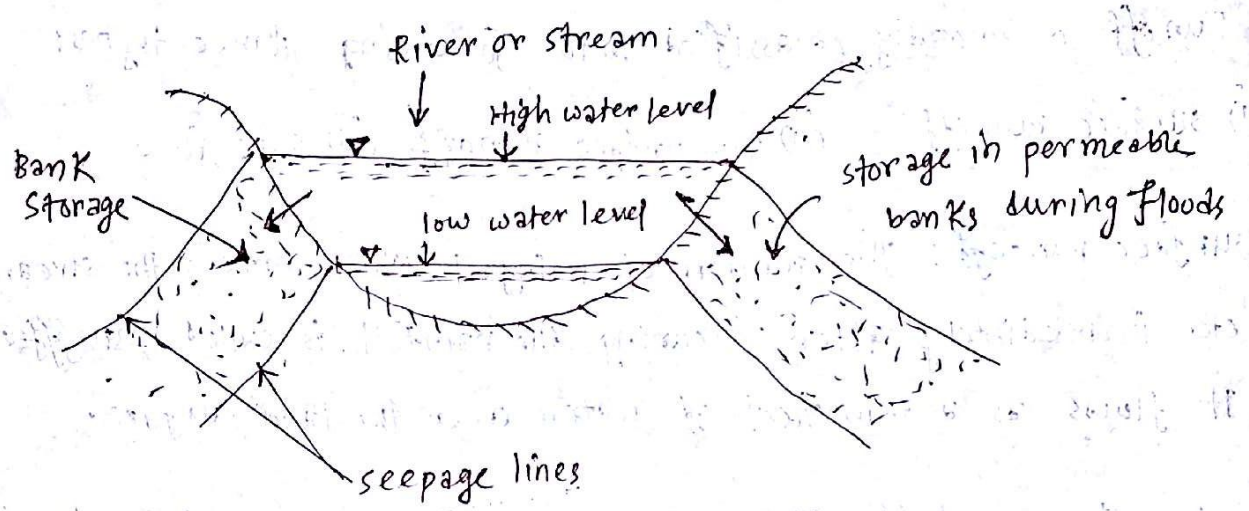
Detention storage: The volume of water in transit, the overland flow which has not yet reached the stream channel is called surface detention or detension storage.

13, 16

Bank storage: The portion of runoff in a rising flood in a stream which is absorbed by the permeable boundaries of the stream above the normal phreatic surface, is called Bank storage.



Catchment characterestics:

07, 17

1. Drainage Basin: The entire area of a river basin whose surface run-off due to storm drains in to the river in the basin is considered as a hydrological unit and is called drainage basin/ watershed/ catchment area of the river flowing.

2. Drainage Divide: The boundary line, along a topographic ridge, seperating two adjacent drainage basins is called drainage divide.

3. concentration point or measuring point: The single point or location at which all surface drainage from a basin comes together or concentrates as out flow from the basin in the stream channel is called concentration point.

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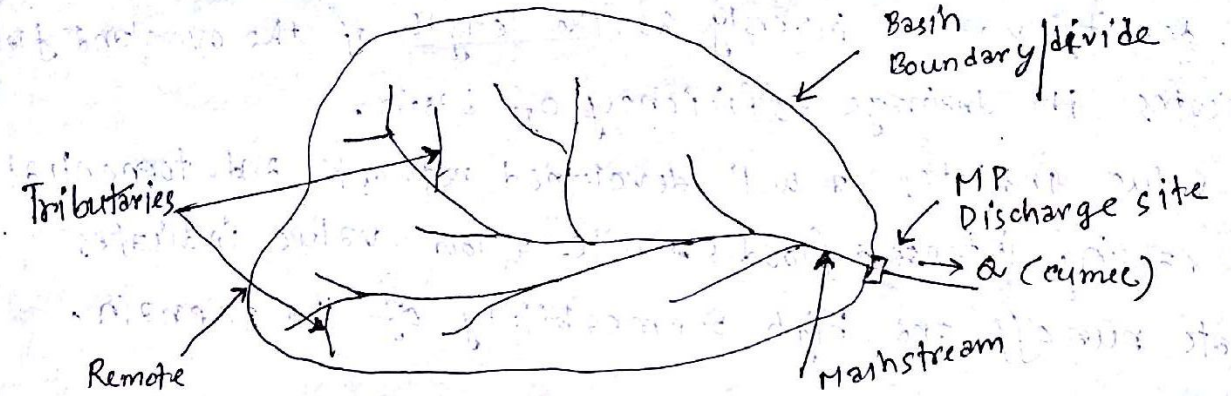

Fig. Drainage Basin characterestics

4. Concentration Time: The time required for the rain falling at the most distant point in a drainage area i.e. on the fringe of the catchment, to reach the concentration point is called concentration time.

Characterestics of Drainage net:

The characterestics of Drainage net may be physically described by:

(i) The number of stream. (ii) The length of streams.

(iii) Stream density. (iv) Drainage density.

Stream density: The stream density is expressed as the number of streams per unit area of basin.

$$\text{stream density, } D_s = \frac{N_s}{A}$$

Drainage density: Drainage density is expressed as the total length of all stream channels (perennial and intermittent) per unit area of the basin, and serves as an index of the areal channel development of the basin.

$$\text{Drainage density, } D_d = \frac{L_s}{A}$$

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Drainage density varies inversly as the length of the overland flow and indicates the drainage efficiency of basin.
A high value indicates a well developed network and torrential runoff causing intense flood s while a low value indicates moderate runoff and high permeability of the terrain.

Average stream slope: Average stream slope is defined as follows:

$$\text{Average stream slope} = \frac{\text{Total fall of the longest water course}}{\text{length of the longest water course}}$$

Horton suggested a formula to compute the slope of basin:

$$\text{Slope of basin, } S = \frac{1.5\,(e_I)\,N_e}{\Sigma L}$$

where, e_I = contour interval
N_e = Number of contours crossed by all the subdividing lines.
ΣL = Total length of the subdividing lines.

Ground water divide: The line of the ground water table from which the water table slopes downward away from the line on both sides, is called the ground water divide.

Shape of a Drainage Basin:

The shape of a drainage basin can generally be expressed by:

(i) Form factor.
(ii) Compactness co-efficient.

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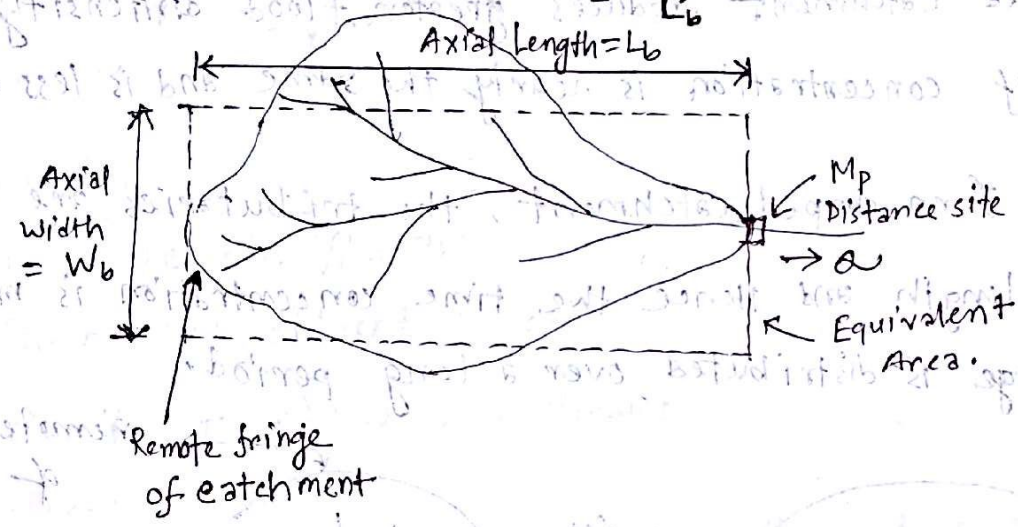
Form factor: It is defined as the ratio of axial width of basin to the axial length of basin.

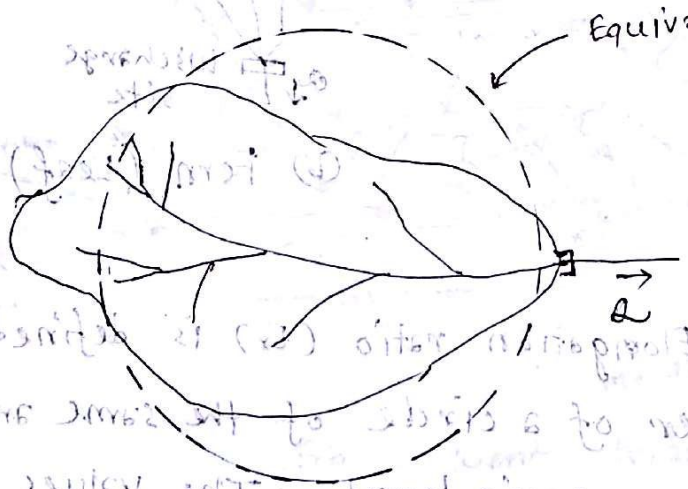
Form factor, $F_f = \frac{W_b}{L_b}$; $A = W_b L_b$

$= \frac{A}{L_b^2}$



Compactness co-efficient: It is defined as the ratio of perimeter of the basin to the circumference of circular area which equals the area of the basin.

Compactness co-efficient, $C_c = \frac{P_b}{2\sqrt{\pi A}}$ / $C_c = \frac{P_b}{2\pi R}$



Equivalent circular area

$A = \pi R^2$

$R = \sqrt{\frac{A}{\pi}}$

$\therefore 2\pi R = 2\pi\sqrt{\frac{A}{\pi}}$

circumference of equivalent circular area $= 2\sqrt{\pi A}$

The compactness co-efficient is independent of the size of the catchment and is dependent only on the slope.

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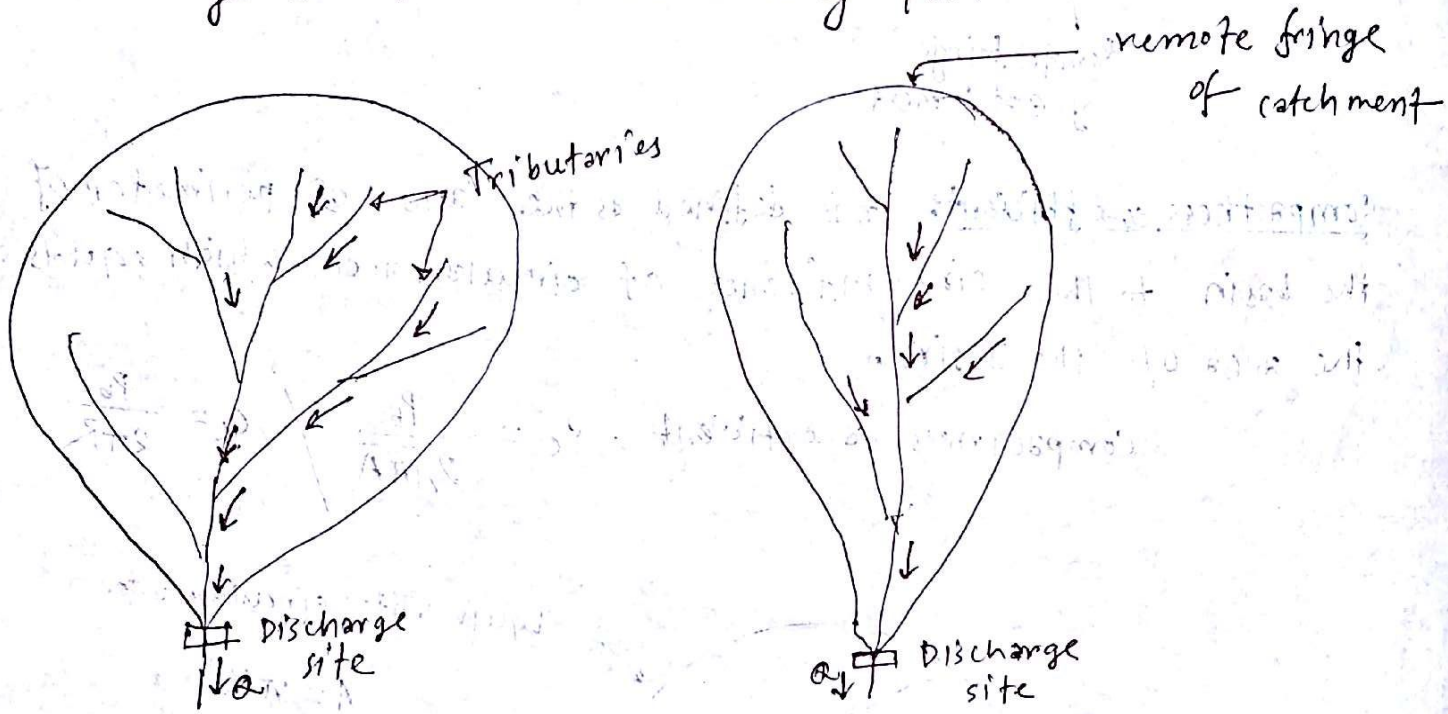
In fern-shaped catchment the concentration time is more than that of fan shaped catchment

Explanation: since all the tributaries are nearly of the same length, a fan shaped catchment produces greater flood intensity. Hence the time of concentration is nearly the same and is less.

Incase of fern shaped catchment, the tributaries are of different length and Hence the time concentration is more and discharge is distributed over a long period.



(a) Fan Shaped (b) Fern (Leaf) shaped.

Elongation Ratio: Elongation ratio (Er) is defined as the ratio of the diameter of a circle of the same area as the basin to the maximum basin length. The values range from 0.4 to 1.0.

$$Er = \frac{2R}{L_b}$$

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Circularity ratio: Circularity ratio (C_r) is defined as the ratio of the basin area to the area of a circle having the same perimeter as the basin. The values range from 0.2 to 0.8.

$$C_r = \frac{A}{\pi R'^2} \quad \text{where, } R' = \frac{P_b}{2\pi}$$

Classification of Stream:

Stream may be classified as:

(i) Influent and Effluent streams

(ii) Intermittent and perennial streams

12, 17, 16, 13

Influent stream: If the GWT is below the bed of the stream, the seepage from the stream feeds the ground water resulting in the build up of water mound. such streams are called influent streams.

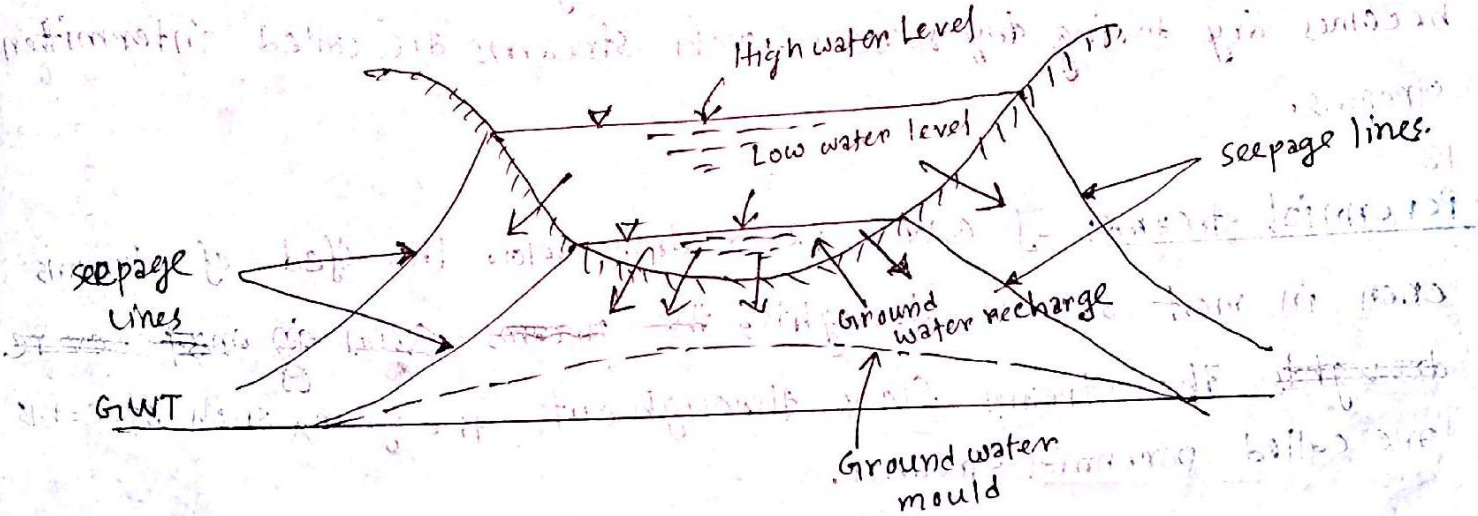

Fig. Influent stream

Influent streams dry up completely in rainless period and are called emphemal streams.

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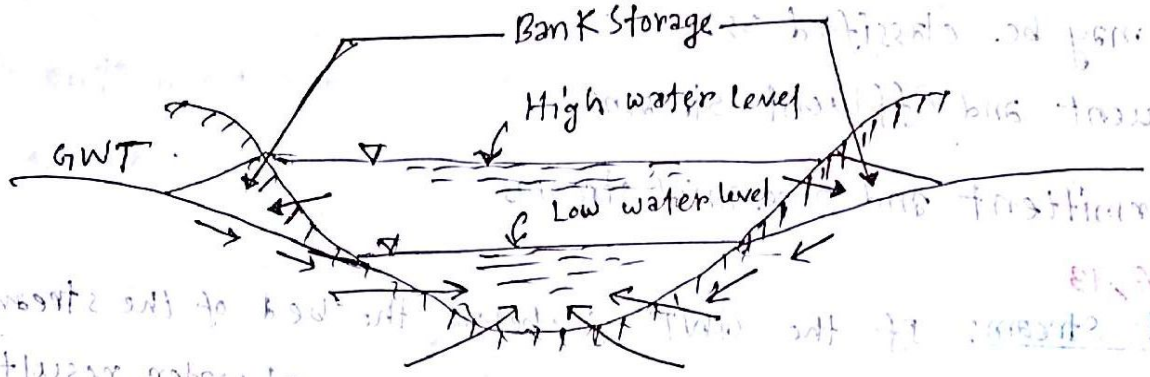
Effluent streams: When the GWT is above water surface elevation in the stream, the ground water feeds the stream. such streams are called effluent streams.

The base flow of surface streams is the effluent seepage from the drainage basin. Most of the perennial streams are mainly effluent streams.



13

Intermittent streams: If the GWT lies above the bed of the streams during wet season but drops below the bed during the dry season, the stream flows during wet season, but becomes dry during dry seasons, such streams are called intermittent streams.

13

Perennial streams: If GWT never drops below the bed of streams even in most severe droughts,

The stream flows throughout the year of such streams are called perennial streams.

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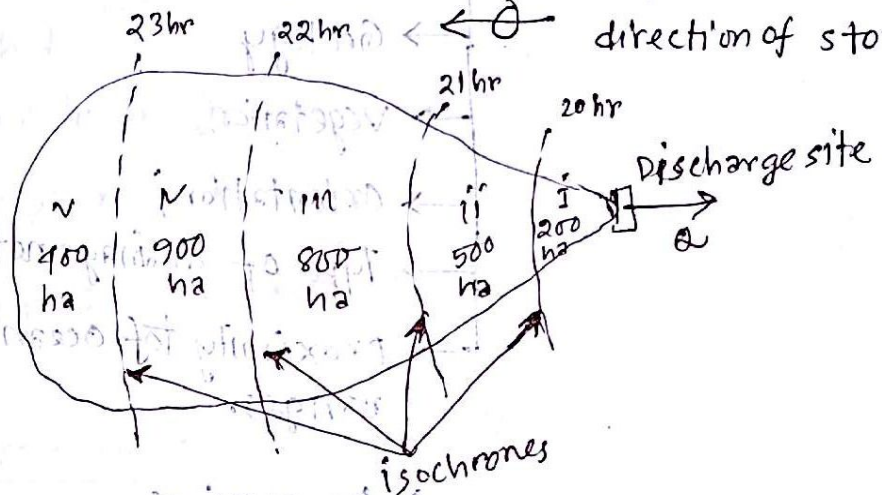
07,13,17,16

ISOCHRONES:

The lines joining all points in a basin of some key time Elements in a storm, such as beginning of precipitation, are called isochrones. They are the time contours and represent lines of equal travel time and they are helpful in driving hydrographs.



Factors affecting Run off:

The various factors, which affect the run off from a drainage basin depend upon the following characterestics:

17.

(i) Storm characterestics →
- Type or nature of storm and season
- Intensity
- Duration
- Areal extent
- Frequency
- Anticedent precipitation
- Direction of storm movement

(ii) Meteorological characterestics →
- Temperature
- Humidity
- Wind velocity
- Pressure variation

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17, 16, 12, 11, 10

(iii) Basin characterestics
- Size
- Shape
- Slope
- Altitute
- Topography
- Geology
- Vegetation
- Orientation
- Type of drainage net
- proximity to ocean and mountain ranges.

(iv) Storage characterestics
- Depressions
- pools and ponds / lakes
- stream
- channels
- check dams
- Upstream reservoir / tanks
- Flood plains, swamps.

07

Maximum flood discharge:

It is the discharge in times of flooding of the catchment area ie when the intensity of rainfall is greatest and the condition of the catchment regarding humidity is also favourable for an appreciable run off.

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Estimation of Run-off:

The runoff from the rainfall may be estimated by the following methods:

(i) Empirical formulae, curves and tables
(ii) Infiltration method.
(iii) Rational method.
(iv) Overland flow hydrograph.
(v) Unit hydrograph method.
(vi) Coaxial Graphical Correlation and API

Rational Method estimation:

A rational approach is to obtain the yield of a catchment by assuming a suitable run-off co-efficient.

$$\text{Yield} = CAP$$

where, C = runoff co-efficient
A = Area of catchment
P = Precipitation

The value of run-off co-efficient varies depending upon the soil type, vegetation, geology etc.

In the rational method, the drainage area is divided in to a number of sub-areas and with known times of concentration for different subareas the runoff contribution from each area is determined.

The choice of the value of C for different sub areas is an important factor in runoff computation by this method.

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Distribution Co-efficient:

The distribution co-efficient can be defined as the ratio of the maximum rainfall at a point to the mean rainfall of the concern catchment.

$$C_d = \frac{\text{Maximum rainfall amount}}{\text{Mean rainfall}}$$

The effect of rainfall distribution on runoff can be presented by distribution co-efficient.

Hydrograph

Hydrograph:

A hydrograph is a graph showing variation of discharge (i.e. stream flow at the concentration point) with respect to time.

03, 05, 06, 08, 11, 13

Various components of a single peaked hydrograph:

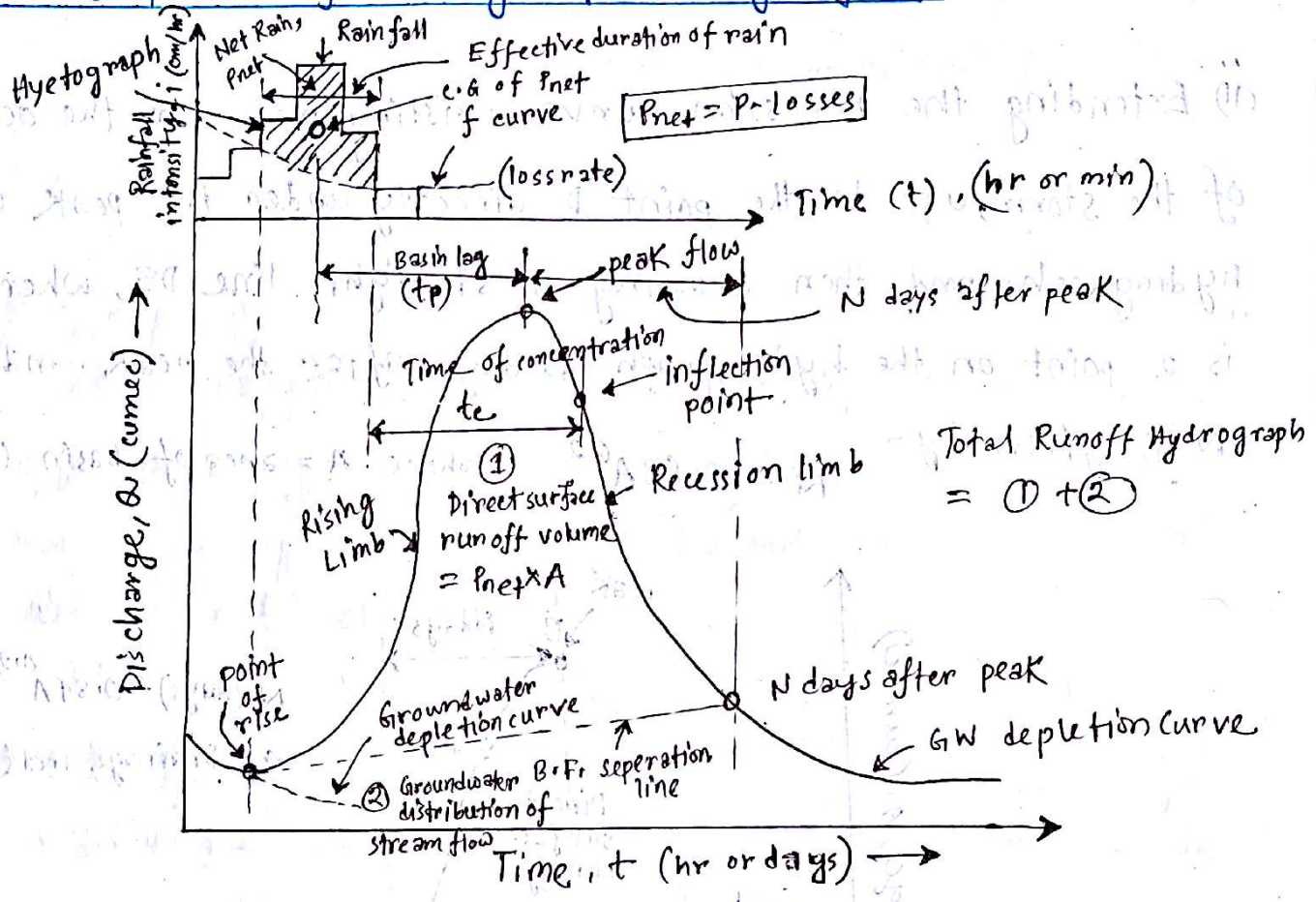

Fig. Components of single peaked hydrograph.

17, 16, 15, 14, 13, 11, 10, 07, 05

Methods of separation of base flow from hydrograph of run off:

For the derivation of unit hydrograph, the base flow has to be separated from the total runoff hydrograph.

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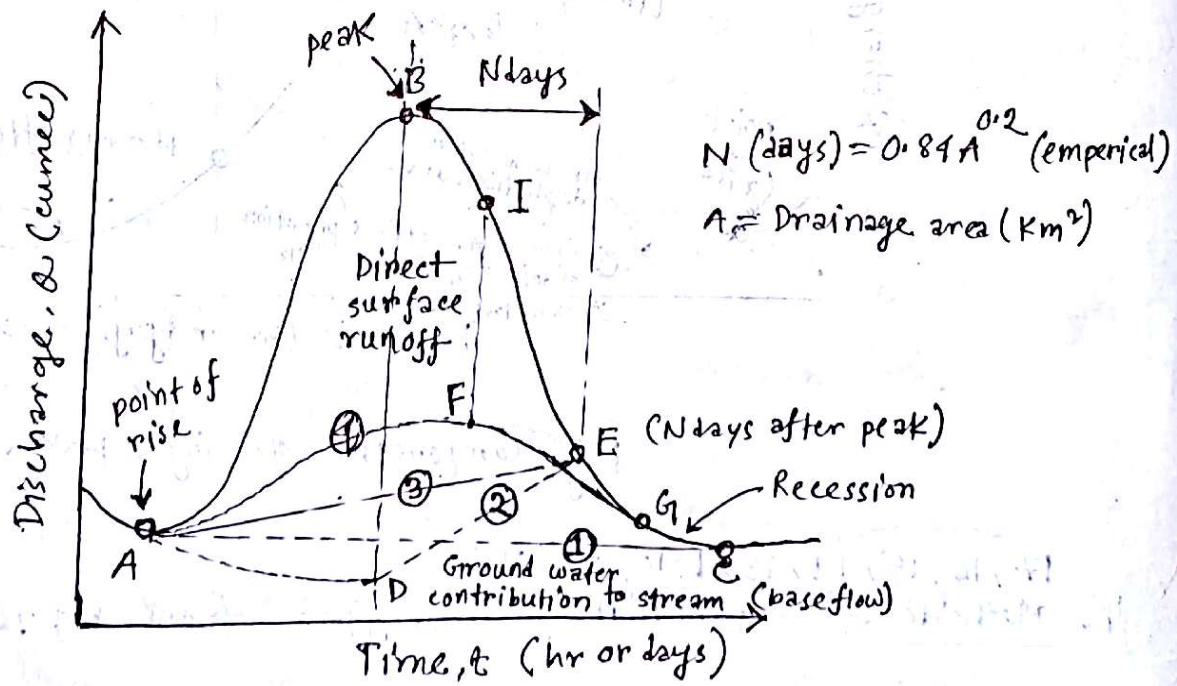
some of the well known base flow seperation procedure are given below,

(i) simply by drawing a line AC tangential to the both limbs at their lower portion. This method is very simple but is approximate and can be used only for preliminary estimates.

(ii) Extending the recession curve, existing prior to the occurence of the storm, up to the point D directly under the peak of the hydrograph and then drawing a straight line DE, where E is a point on the hydrograph N days after the peak. and N is given by -

$$N = 0.83A^{0.2}$$ where A = area of basin (km^2)



(iii) simply by drawing a straight line AE, from the point of rise to the point E, on the hydrograph, N days after peak.

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(iv) construct a line AFG by projecting backwards the ground water recession curve after the storm, to a point F directly under the inflection point of the falling limb and sketch an arbitrary rising line from the point of rise of the hydrograph to connect with the projected base flow recession.

This type of seperation is preferred when the ground water is relatively large and reaches the stream fairly rapidly, as in lime stone terrains.

Unit Hydrograph:

The unit hydrograph is defined as the hydrograph of storm runoff resulting from an isolated rainfall of some unit duration occuring uniformly over the entire area of the catchment; produces a unit volume of runoff.

Elements of unit hydrograph:

Base width (T) – The period of direct surface runoff (due to a unit storm) of the unit hydrograph is called the base width.

Unit storm – The storm of unit duration regardless of its intensity is called unit storm.

Unit period:– The time duration of the unit storm is called unit period.

Lag time (t_p) – The time from the centre of a unit storm to the peak discharge of the corresponding unit hydrograph is called lag time.

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Recession time (T_r) – The duration of the direct surface runoff after the end of the excess or net rainfall, is called recession time in hydrograph analysis.

2012, 08

Propositions of the unit hydrograph:

The follow are the basic proposition of the unit hydrograph:

(i) Same runoff duration: For all unit storms of different intensities, the period of surface runoff is approximately the same, although they produce different runoff volumes.

(ii) Proportional ordinates: For unit storms of different intensities, the ordinate of hydrograph at any given time, are in the same proportion as the rainfall intensities.

(iii) principle of superposition: If there is no continuous storm or isolated storms of uniform intensity net rain, they may be divided into unit storms and hydrographs of runoff for each storm obtained, and the ordinates added with the appropriate time lag to get combined hydrograph.

(iv) Same distribution percentage: If the total period of surface runoff is divided into equal time intervals, the percentage of surface runoff that occurs during each of these periods will be same for all unit storms of different intensities.

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17, 15, 14, 13, 12, 08, 06, 05, 03

Limitations of Unit hydrograph:

certain limitations are inherent in the unit hydrograph theory.

(i) The runoff hydrograph reflects the combined effects of rainfall factors, loss factors and physiographic factors.

(ii) The unit hydrograph can be applied only to drainage basins with small areas.

(iii) The unit hydrograph can not be applied for basins larger than 5000 km^2.

(iv) The unit hydrograph is not suitable for areas less than 200 hectares.

(v) The unit hydrograph are not well adopted to basins with odd shapes, particularly those which are long and narrow.

(vi) The principle of linearity is not valid strictly.

(vii) The unit hydrograph method can not be applied when a major portion of storm precipitation is in the form of snow.

Assumptions of unit hydrograph theory:

(i) The effective rainfall is uniformly distributed within its duration.

(ii) The effective rainfall is uniformly distributed throughout the whole area of the basin.

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(c) The base periods of the direct runoff hydrographs produced by effective rainfall of same duration are also same.

(d) The ordinates of the direct runoff hydrographs of a common base period are directly proportional to the total volume of direct runoff

(e) For a given drainage basin the hydrograph of runoff due to a given period of rainfall reflects the unchanging characteristics of the basin.

Derivation of unit hydrograph:

1. An isolated storm of some unit period of heavy rainfall is selected using past record of several storms.
2. Hydrograph is developed with the help of runoff data obtained from selected storm.
3. Base flow is seperated using most appropriate method.
4. The ordinate of direct runoff is obtained.
5. The total depth of effective rainfall is calculated by the following equation:

$$\text{Effective rainfall depth (cm)} = \frac{\sum Q_d \cdot t}{A}$$

6. The ordinates of unit hydrograph is determined using following relation:

$$\text{ordinate of unit hydrograph} = \frac{\text{ordinates of direct runoff}}{\text{Effective rainfall depth (cm)}}$$

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7. All obtained ordinates of unit hydrograph with corresponding time is plotted.

The obtained hydrograph is unit hydrograph which has unit area below the curve.

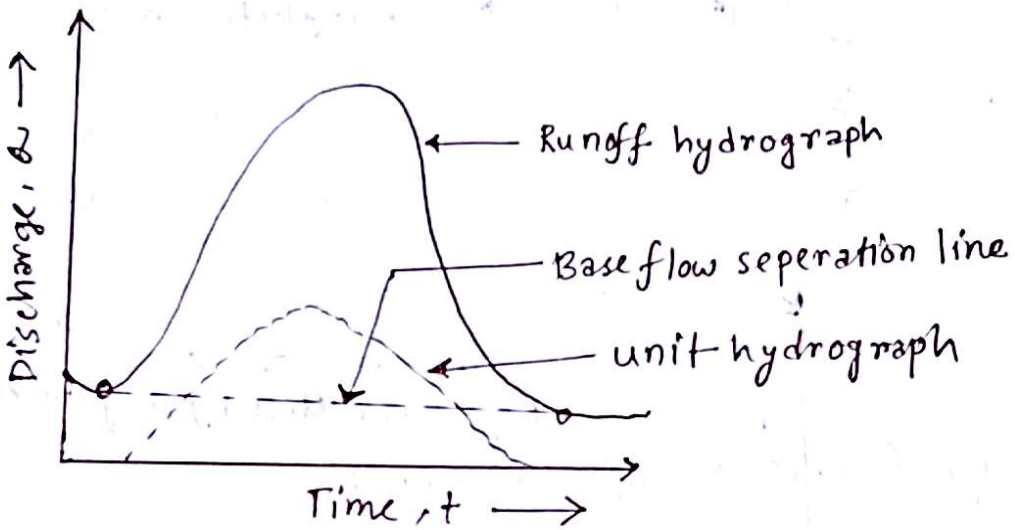

Fig. Derivation of unit hydrograph.

Instantaneous unit Hydrograph:

Instantaneous hydrograph is a hydrograph of runoff resulting from the instantaneous application of 1 cm net rain on the drainage basin. The IUH in conjuction with the design storm can be used to obtain the design flood by using a convolution integral.

6-hours unit hydrograph:

The hydrograph of storm runoff, resulting from an isolated rainfall of 6 hour duration, occuring uniformly over the entire area of catchment, producing a unit volume (1cm) of runoff called 6-hour unit hydrograph.

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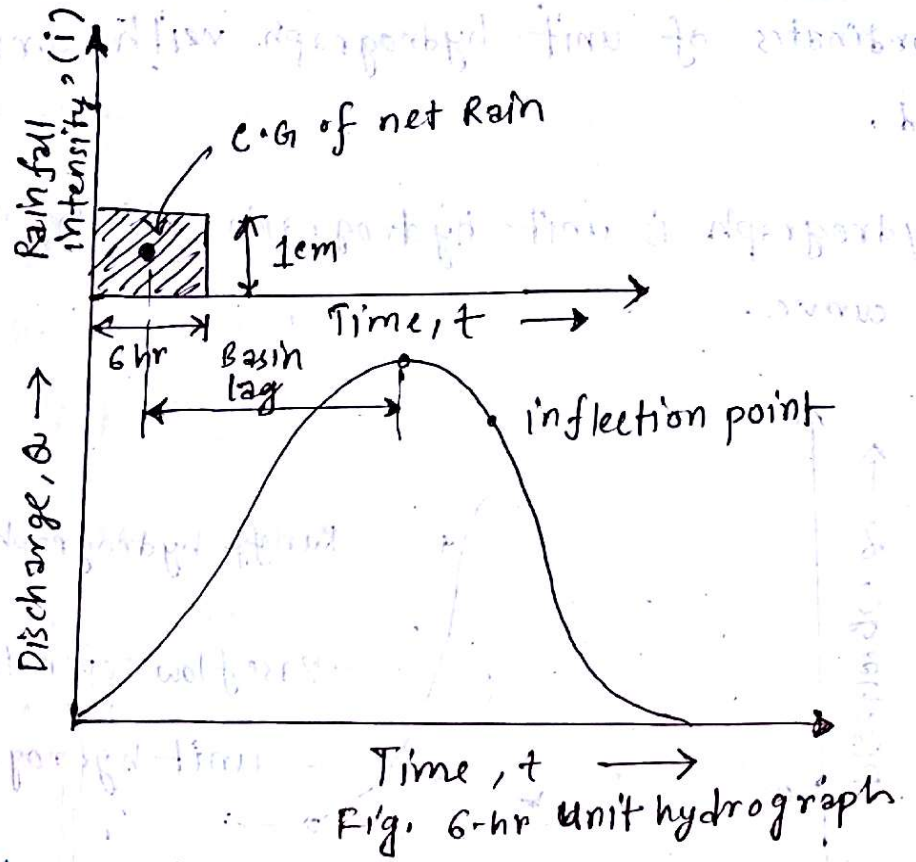


Fig. 6-hr unit hydrograph

Significance of inflection point:

Inflection point is a point on the recession limb where there is a change in slope.

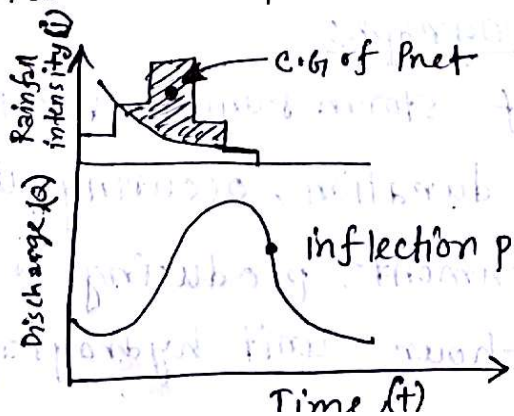
Significance:

(i) It indicates the end surface of inflow to the channel.

(ii) It represent the condition of maximum storage.

(iii) It is the point after which gradual withdrowl of catchment storage is started.

(iv) It is the point after which hydrograph is independent of storm characteristics and dependent only on basin characterestics.



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Stream Gauging

Stream Gauging:

The measurement of discharge of the stream draining it is termed as stream gauging. It is the most satisfactory determination of the runoff from a catchment.

Methods of stream gauging: 05, 02

Some of usual methods of stream gauging are given below:

(i) venturiflumes or standing wave flumes for small channel.
(ii) weirs or anicuts.
(iii) slope area method.
(iv) contracted area method.
(v) sluiceways, spillways and power conduits.
(vi) Salt-concentration method.
(vii) Area-velocity method.

03

* Area velocity method: $Q = AV$

The area of cross-section of flow may be determined by sounding and plotting the profile. The mean velocity of flow (V) may be determined by making velocity measurements.

The method of velocity measurements are as follows:

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velocity measurements from
- Floats : measure surface velocity (v_s)
  $V \simeq 0.85 v_s$
- velocity rods : measure mean velocity (V)
- current meter : measure mean velocity in a strip of cross section

by (i) one point Method.

$V = v_{0.6d}$

(ii) two point Method.

$$V = \frac{v_{0.2d} + v_{0.8d}}{2}$$

(d = mean depth in the strip)

current meter Rating curve: 09, 08, 06, 05, 02

The relation between revolution per second (N) of the current meter and velocity of flow past the meter is given by the rating equation,

$V = aN + b$ where, a and b are constant determined from current meter.

The process of calibration of the current meter is called rating of current meter.

A current meter rating curve is drawn by plotting as 'pushing velocity vs. rps' of the meter which plots a straight line.

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* preparation of current rating curve: 09, 08, 06, 05, 02

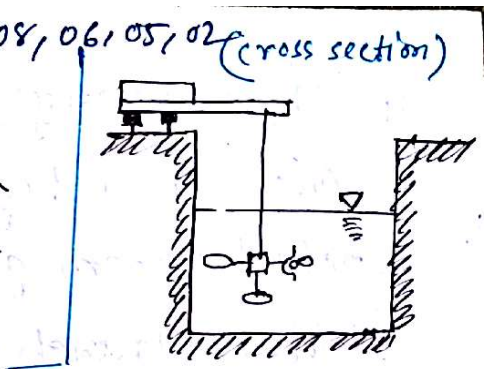
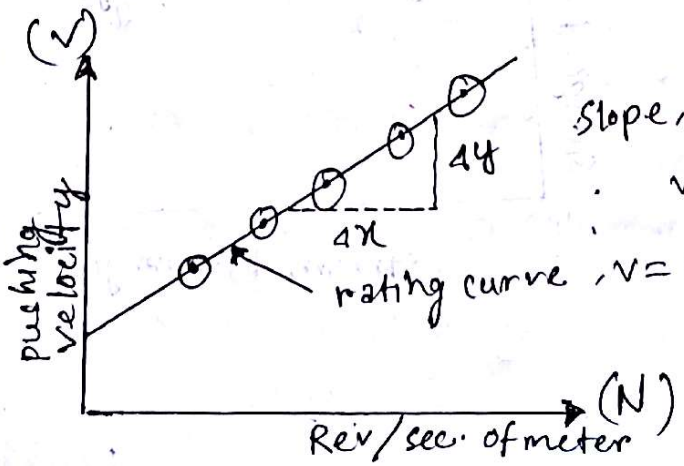


Fig. current meter rating tank

1. The rating is done in a masonry tank
2. The meter is towed in still water by –
   (i) pushing trolley by men or,
   (ii) electric motor.
3. The meter should be immersed about 1 m below the water surface during rating
4. A metronome is adjusted to give ticks at every 1 to 10 seconds and is kept on the trolley.
5. The trolley is pushed uniformly to synchronise with ticking metronome.
6. The observer sits in the trolley, notes time and distance traversed by trolley for fixed number of revolution made by meter.
7. And A current meter rating curve is drawn by plotting 'pushing velocity with respect to rps'. The curve is a straight line. Constants 'a' and 'b' can be determined from the curve



Slope, $a = \frac{\Delta y}{\Delta x}$

$V = \frac{\text{distance towed (m)}}{\text{time (sec)}}$

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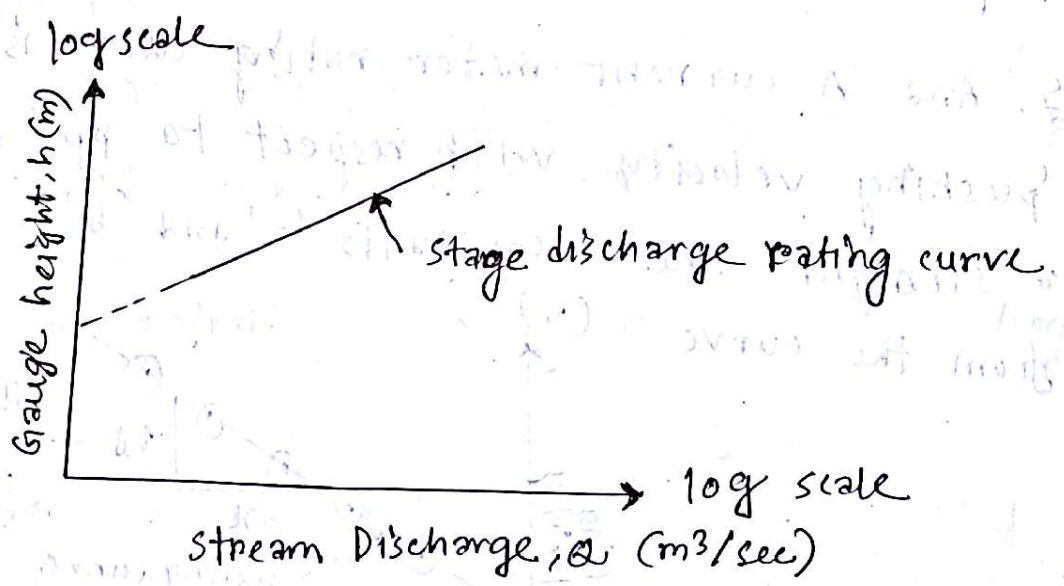
Stage discharge rating Curve:

Once the rating equation of the current meter is known, actual stream gauging can be done from bridges, cradle, boat or launch.

The cross section of the stream at gauging site is divided into elemental strips of equal width b and current meter is lowered to a depth of 0.6d below water surface in shallow depths (One point method) and to depth 0.2d and 0.8d (two points method) in deep water, at the centre of each strip. The mean depth (d) at the centre of each strip is determined by sounding.

The velocities at the approximate depths are determined from the rating equations. And The discharge in each elemental strip is determined. The discharge in the stream is the sum of the discharges in all the elemental strips.



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Discharge determination method in each elemental strip:

There are two method of determining the discharge in each elemental strip:

(i) Mid-section method: In this method, the vertical in which the velocity measurements are made (by one point or two points method) is taken as the middle of the strip and the water depth (d) in the vertical is taken as the mean depth of the strip. If b is the width of strip then,

$$\Delta Q = (bd) v_{0.6d} \quad \text{in shallow strips}$$

$$\Delta Q = (bd) \times \frac{v_{0.2d} + v_{0.8d}}{2} \quad \text{in deep water strips.}$$

Stream discharge, $Q = \sum \Delta Q$

In this method, the discharge in the two triangular bits near the ends are not included in discharge computation.

(ii) Mean section method: In this method the elemental strip is taken between two verticals and the mean depth is taken as the average of the depths in the two verticals. The width of the the strip is distance 'b' between two verticals. The discharge in the elemental strip is given by -

$$\Delta Q = b\left(\frac{d_1+d_2}{2}\right)\left(\frac{v_1+v_2}{2}\right)$$

and, stream discharge, $Q = \sum \Delta Q$

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Adjustment of stage Discharge rating curve:

When discharge measurements are made during both rising and falling stages and also at constant stages, the points joining the rising and falling stages plot a loop due to channel storage and variation of water surface slope as a flood wave moves along the stream.

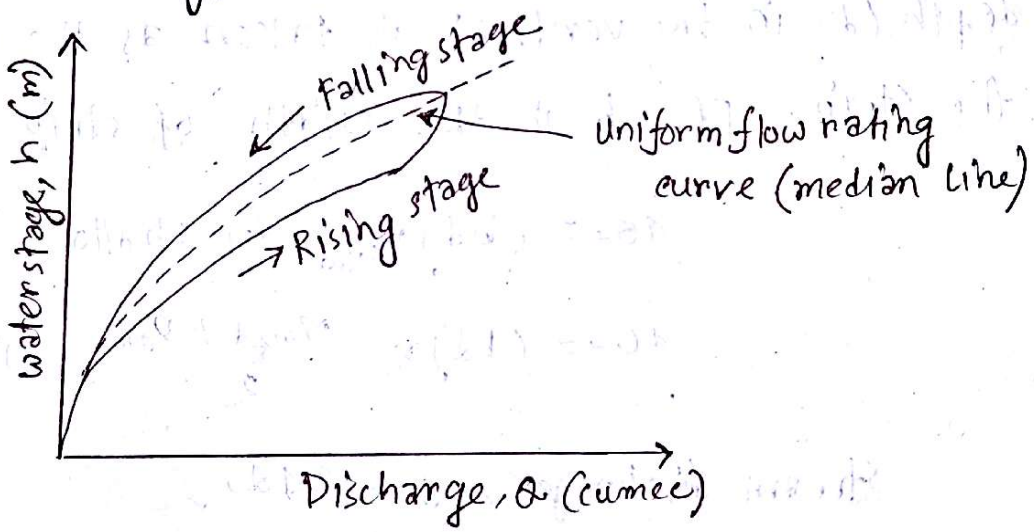

Fig. Adjustment of flow rating curve

A smooth discharge curve is drawn along the median curve line passing through or near the points that were obtained at constant stage.

Correction has to be applied for the discharge obtained from the rating curve during a rising or falling stage i.e. during a flood. For this purpose, an auxiliary gauge is established some distance upstream or down stream from the main gauge.

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Then, $\frac{Q_a}{Q_o} = \left(\frac{\Delta h_a}{\Delta h_o}\right)^n$ ..... ①

where, $n = 0.5$, since Q varies as the square root of the energy gradient.

From the equation ① the actual discharge Q_a during a rising or falling stage can be determined.

Selection of site for a stream gauging station: 09, 08

The following factors have to be considered in selecting a site for a steam gauging station:

(i) The seleection should be straight and uniform for a length of about 10 to 20 times the width of the stream.

(ii) The bed and banks of the stream should be firm and stable so as to ensure area-discharge consistancy.

(iii) The bed and banks should be free from vegetal growth, boulders or other obstructions.

(iv) There should be no larger over flow section at flood stage.

(v) To ensure consistancy between stage and discharge, there should be a good control section far down stream of gauging site.

(vi) The site should avoided from influencing zone of back water of dam.

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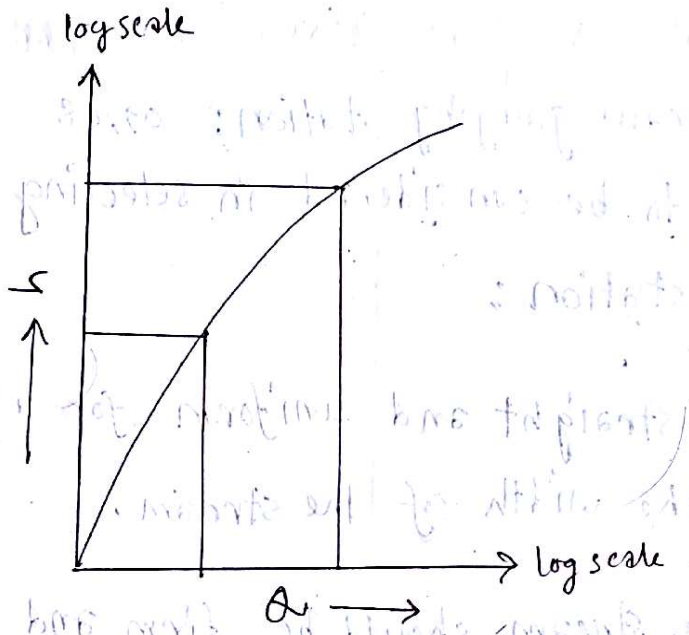
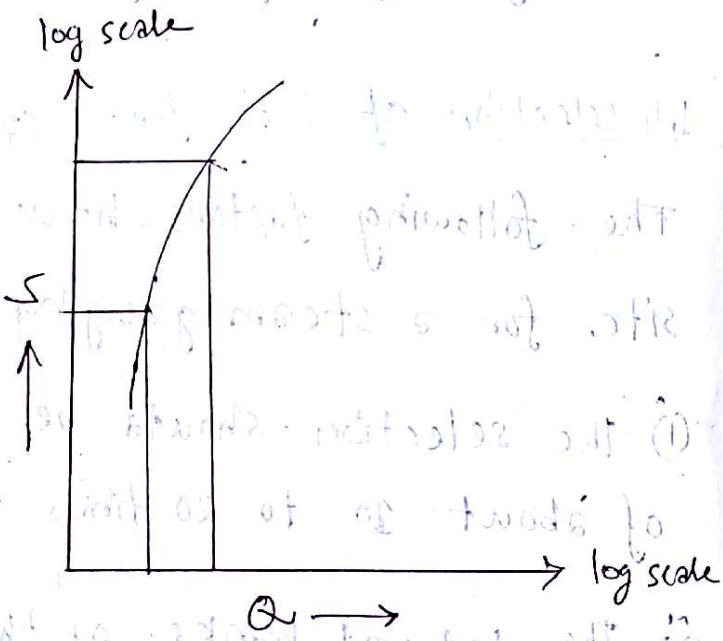
(vii) The stream gauging station should be easily accessible.

(viii) The site should be well protected.

Qualatative diagram of stage discharge rating curve of Big river and small rive:





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Flood Estimation and Control

Flood:

A flood is an unusual high stage of a river due to runoff from rainfall and melting of snow in quantities too great to confined in the normal water surface elevations of river or stream as the result of unusual meteorogical combination.

Design Flood: 2015, 13

The maximum flood that any structure can safely pass is called the 'Design flood'. It is adopted for design of hydraulic structures like spillways, bridge openings, flood banks etc, Design flood is usually selected after cost benifit analysis.

Reference in the design flood estimates:

In design flood estimates, Reference is usually made to three classes:

1. Standard Project Flood (SPF): 2015

The estimation of the flood likely to occur from the most severe combination of the meteorogical and hydrological conditions, but excluding extremly rare combination is called Standard Project Flood.

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2. Maximum Probable Flood (MPF): 2015, 13

It includes the extremely rare and catastrophic floods and is usually confined to spillway design of very high dams. The SPF is usually around 80% of the MPF for the basin.

3. Probable maximum Precipitation (PMP):

The moisture flow index in the storm is determined by -

1. Observation of air moisture from maximum due point
2. Temperature recorded and
3. Air inflow.

The best known upward adjustment to be applied to the historical and hypothetical major storms in the maximisation with respect to moisture change. From the critical combination of storms and moisture adjusted, probable maximum precipitation is derived.

After minimizing losses, PMP can be applied on the design unit hydrograph to produce MPF.

Estimation of Peak flood:

The maximum flood discharge (peak flood) in a river may be determined by the following methods:

1. Physical indication of past floods.
2. Empirical formulae and curves.
3. concentration time method.

Scanned by CamScanner

4. Overland flow hydrograph.
5. Rational Method.
6. Unit Hydrograph.
7. Flood frequency studies.

* Concentration Time Method: The concentration time method of estimating the peak discharge consists of two steps;

(i) Determination of concentration time

(ii) selection of the period of maximum net rainfall for the concentration time duration.

This method can be used for design storms or in conjuctions with intensity duration frequency curves.

2015

* Rational Method: The rational Method is based on the application of the formula;

$$Q = CiA$$

where, C is a co-efficient depending on the run off qualities of the catchment called run-off co-efficient (0.2 to 0.8)

i = intensity of rainfall = i_c can be found from the intensity duration frequency curves for the catchment corresponding to t_c and T.

If the curves are not available, the design intensity can be obtained from the equation,

$$i_c = \frac{P}{t_R}\left(\frac{t_R+1}{t_c+1}\right)$$

When the time of concentration, t_c is not known, $i_c \approx \frac{P}{t_R}$

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Frequency Factor:

The magnitude of the flood,

$$x_T = \bar{x} + K_T \cdot S_x$$

where, $\bar{x}$ = mean

S_x = standard deviation.

K_T = Frequency factor

Frequency factor is given by,

$$K_T = \frac{y_T - \bar{y}_n}{\sigma_n}$$

The frequency factor K_T depend on the type of distribution and return period.

Return Period: 08, 06

Return period indicates the average number of years with in which a given event will be equalled or exceeded.

Formula: California formula:

$$T_n = \frac{n}{m} = \frac{1}{P(x_m)}$$

where, $P(x_m)$ = Exceedence probability

Hazen's formula:

$$T_r = \frac{2n}{2m-1}$$

Weibul's formula: $T_r = \frac{n+1}{m}$

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Procedure to estimate the design flood for any return period, using Gumbel's distribution:

1. From the given data on flood peaks for n years, the mean $\bar{x}$ and standard deviation S_x are computed by the following equations:

$$\bar{x} = \frac{x_1 + x_2 + \cdots + x_n}{n} \quad \text{and} \quad S_x = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n-1}}$$

2. For the known sample size n, the values of $\bar{y}_n$ and σ_n are obtained.

3. From the given return period T_r, value of y_T is found from the following equation:

$$y_T = -\ln\left[\ln\left(\frac{T_r}{T_r - 1}\right)\right]$$

4. Frequency factor K_T is found from the following equation:

$$K_T = \left(\frac{y_T - \bar{y}_n}{\sigma_n}\right)$$

5. Now design flood x_T is found by the following equation:

$$x_T = \bar{x} + K_T \cdot S_x$$

Flood Routing

Determination of Muskingum Parameters K and x:

When a pair of observed inflow and outflow hydrograph is avaiable for a reach, the values of K and x of the reach can be determined using these graphs.

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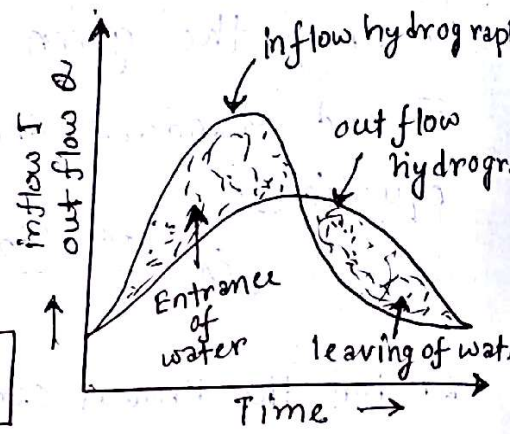
when these two graphs are plotted it will be seen that both cross on the recession side of the inflow hydrograph.

The routing equation,

$$I - Q = \frac{ds}{dt}$$

and, $S = K[xI + (1-x)Q]$

$$\therefore \frac{ds}{dt} = K\left[x \cdot \frac{dI}{dt} + (1-x)\frac{dQ}{dt}\right]$$



Hence, $$I - Q = K\left[x \cdot \frac{dI}{dt} + (1-x)\frac{dQ}{dt}\right] \quad \cdots\cdots (1)$$

when, the two hydrograph are crossing $I = Q$ and $I - Q = 0$

Making use of condition of this condition in Equation (1) we obtain,

$$x = \frac{\frac{dQ}{dx}}{\frac{dQ}{dx} - \frac{dI}{dt}} \quad \cdots\cdots (11)$$

Substituting the value of x in equation (1) we obtain the value of K.

The values of K and x determined using this procedure are only approximate since the slope of hydrographs can not be found out precisely.

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Flood Routing

Flood Routing:

Flood routing may be defined as the procedure where by the shape of a flood hydrograph at a particular location on the stream is determined from the known or assumed flood hydrograph.

Uses of Flood Routing:

Flood routing is used in—

1. Determining required levee heights for flood protection.
2. Estimating the height of a flood peak at a downstream location in short term flood forecasting.
3. Estimating the protection that would result from construction of a reservoir.
4. Determining the adequacy of spillways.
5. Predicting the behavior of a river after a change in channel condition.
6. the derivation of synthetic unit graphs.

Difference Between channel Routing and Reservoir routing:

Channel Routing	Reservoir Routing
(1.) It analyses the effect of storage of specified channel reach on the flood hydrograph.	(1.) It analyses the effect of reservoir storage on the flood hydrograph.
(2.) This gives an information about flood occurence in the river.	(2.) For this purpose, Information such as: storage vs. elevation curve, outflow vs. elevation curve, inflow hydrograph are required.

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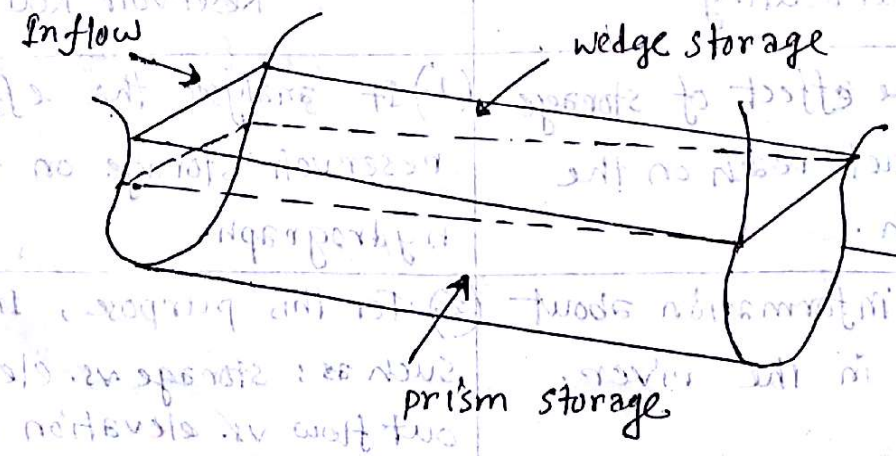
Channel Routing	Reservoir Routing
(3) Some channel routing methods are: (a) Muskingum method. (b) Muskingum crest segment method. (c) Graphical method.	(3) Some reservoir routing methods are: (a) I. S. D method (b) Modified puls method (c) Trial and error method (d) Graphical Method

Difference between prism storage and wedge storage:

Prism Storage	Wedge storage
(1) The prism storage is formed by a volume of constant cross section along the length of the prismatic channel.	(1) Wedge storage may be taken as a fraction of the volume of the prism corresponding to $(I-Q)$ where I = Inflow, Q = Outflow
(2) Prism storage depends on the out flow alone.	(2) Wedge storage depends on the difference, $(I-Q)$
(3) prism storage = KQ	(3) Wedge storage = $K \cdot x \cdot (I-Q)$



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Ground Water

 # Ground water: In general, The term 'ground water' or 'sub surface water' refers to the water that occurs below the surface of the earth.

The main source of ground water is infiltration. The infiltrated water after meeting the soil moisture deficiency percolates deeply and becomes ground water.

 # Advantages of ground water:

1. Ground water is free from pollutants.
2. Ground water is free from pathogenic organism.
3. It is very useful for domestic use in small towns and isolated farms.
4. It can be made avaiable at a small capital cost and also in least possible time
5. In arid region, Ground water is often the only reliable source of water for irrigation.
6. Ground water is rich in several minerals.
7. It has been an important water resource throughout the ages.

 # Formation of soil below earth surface:

✓ The formation below the earth's surface is divided into two zones by an irregular surface called water table. At all points on the water table, the pressure is atmospheric.

✓ The zone between the ground surface and the water table is called unsaturated zone or vadose zone. In vadose zone the soil pores may contain either air, or water or both. Hence it is also called zone of aeration. water in this zone is held to the soil particles by capillary forces. Thus while this water is able to move with in vadose zone, it cannot move out of the zone in to wells or other places that are exposed to the atmosphere.

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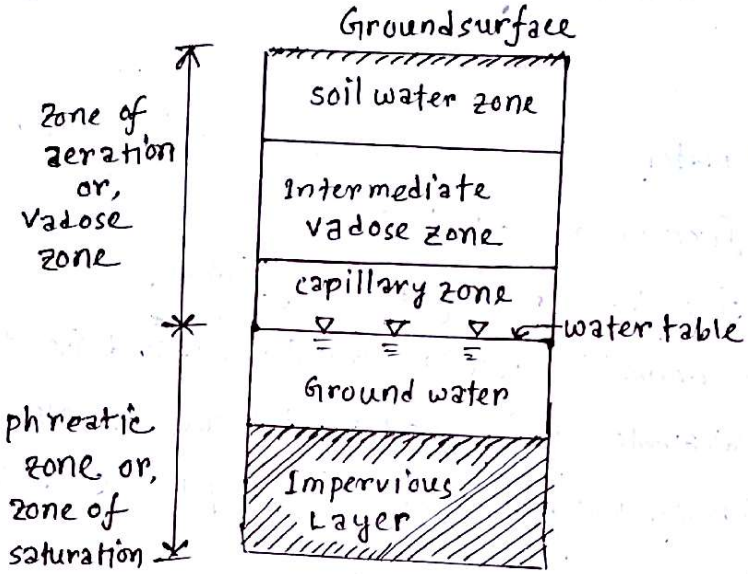
✓In the zone below the water table, all the soil pores are completely filled with water and hence it is called the zone of saturation or the phreatic zone. The phreatic zone may extend to considerable depth, but as the depth increases the weight of overburden tends to close the pore space.



Occurance of Groundwater:

Aquifer: Formations which contain ground water and at the same time which are sufficiently permeable to transmit and yield water in usable quantities are called the aquifers.

The amount of water contained by the aquifer depends on the porosity of the aquifer formation while the amount of water that it can yield depends on its permeability.

Unconfined aquifer: An aquifer having water table in it is called an unconfined aquifer.

For this aquifer water table serves as the upper surface while

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a less permeable or impermeable layer defines its lower boundary. This impermeable layer may be made of clay, shale, solid limestone, igneous rock or bed rock.

Unconfined aquifer is also known as free aquifer, phreatic aquifer, water table aquifer or non artesian aquifer.

confined aquifer: When a aquifer is sandwiched between two layers of much less permeable then it is called confined aquifer.

Example: A sandy layer between two clay layer, sand stone layer between two layers of shale etc.

It is also known as pressure aquifer or artesian aquifer.

They will have recharge area some where on the earth surface.

Piezometer surface: Piezometer surface is the surface obtained by connecting equilibrium water levels in the tubes or piezometer penetrating the confined aquifer.

Artesian well: If the piezometer level at the place of well is above the upper confining layer, the static water level in the well will above the aquifer. such a well is called Artesian well.

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Flowing well: If the piezometric surface at the place of well is above the Ground level, the confined aquifer will yield a free flowing well also known as simply flowing well.

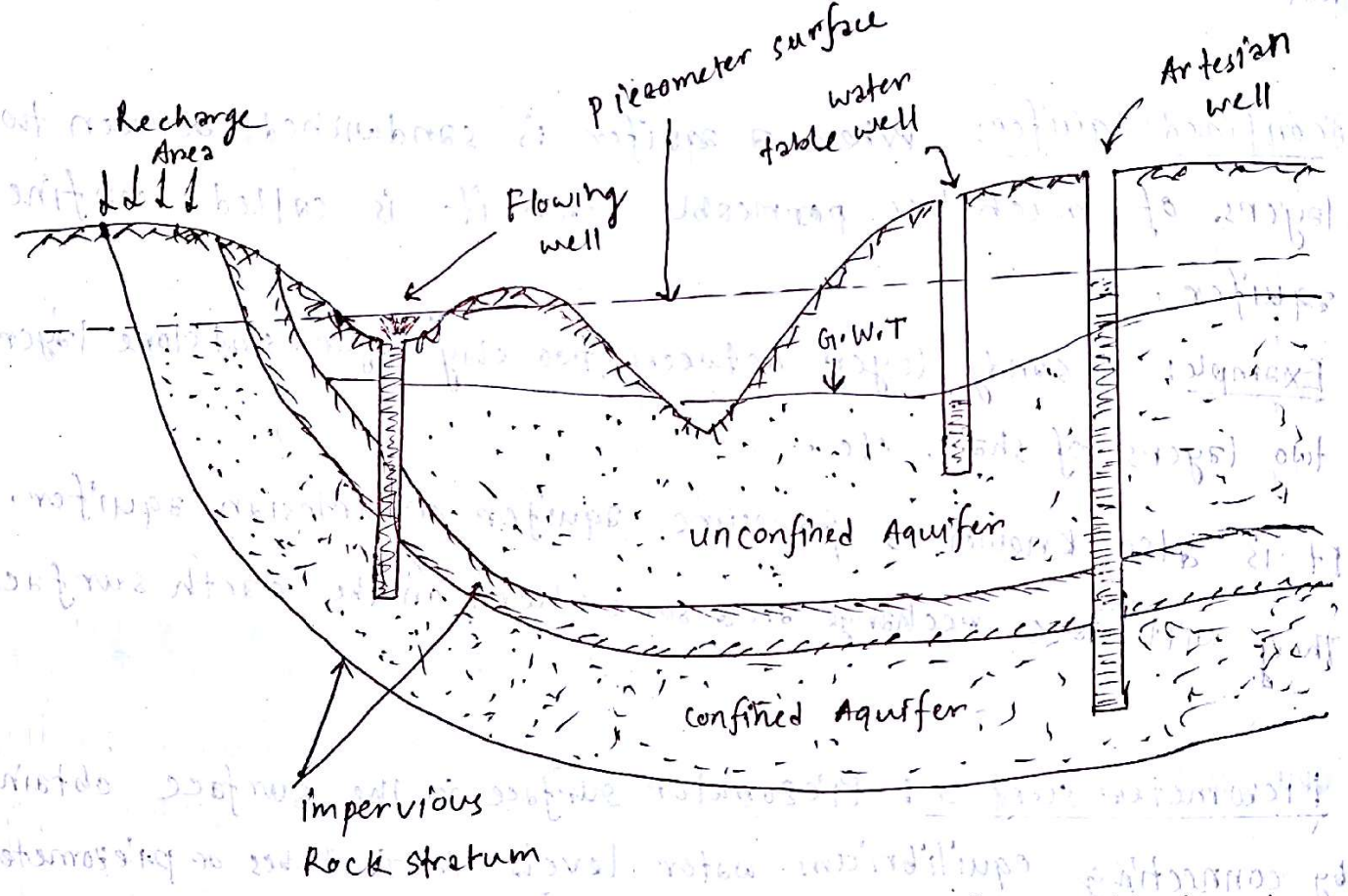

Fig. Ground water in confined and unconfined aquifer.

Aquiclude: A geological formation which is saturated and which may contain large amount of water because of high porosity, but cannot transmit water as it is relatively impermeable is called Aquiclude.

Example: A clay layer

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Aquitard: A saturated geological formation which is poorly permeable hence it does not yield water freely to wells. It may transmit vertically appreciable amount of water from adjacent aquifers.

Example: Sandy clay layer.

Aquifuge: An impervious geological formation which neither contains nor transmits water is known as an aquifuge.

Example: Solid granite rock.

Leaky aquifer: An aquifer bound by one or two aquitards is called a leaky aquifer or a semi-confined aquifer.

Perched Ground water: When a ground water body is separated from the main ground water by a relatively impermeable stratum of small areal extent and by the zone of aeration above the main body of ground water, it is called perched ground water.

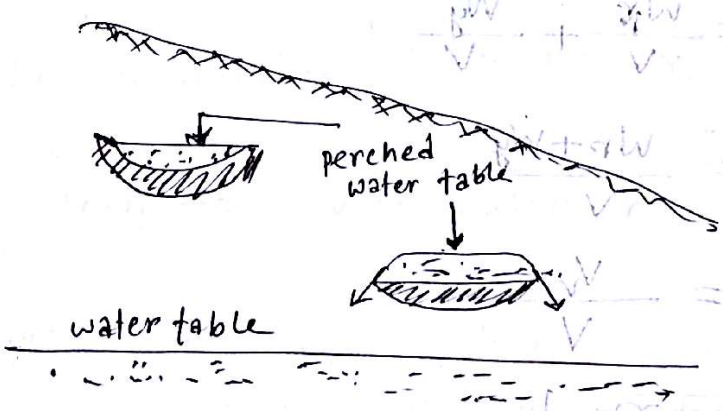

Fig. Perched water table

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Aquifer Parameters:

Porosity: The ratio of volume of pore space (V_v) to the volume of the formation (V) is called the porosity. It is usually denoted by 'n'.

$$n = \frac{V_v}{V}$$

Specific Yield: The specific yield of an aquifer is the ratio of the volume of water which will drain freely from the material to the volume of the formation. It is denoted by 'Sy'.

$$S_y = \frac{W_y}{V}$$

since $W_y < V_v$, $S_y < n$

Specific Retention:

The specific retension S_r is defined as the ratio of the volume of water retained (W_r) in the material to the total volume of material (V), when a saturated material is dewatered.

$$S_r = \frac{W_r}{V} \quad \text{where, } W_r = V_v - W_y$$

$$S_r + S_y = \frac{W_r}{V} + \frac{W_y}{V}$$

$$= \frac{W_r + W_y}{V}$$

$$= \frac{V_v}{V}$$

$$\therefore (S_r + S_y) = n$$

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Storage co-efficient: The storage co-efficient, S is defined as the volume of water that an aquifer releases from or takes in to storage per unit surface area of aquifer per unit drop of water table in the case of an unconfined aquifer and per unit drop of piezometer surface in case of a confined aquifer.

For unconfined aquifer, $S = S_y$

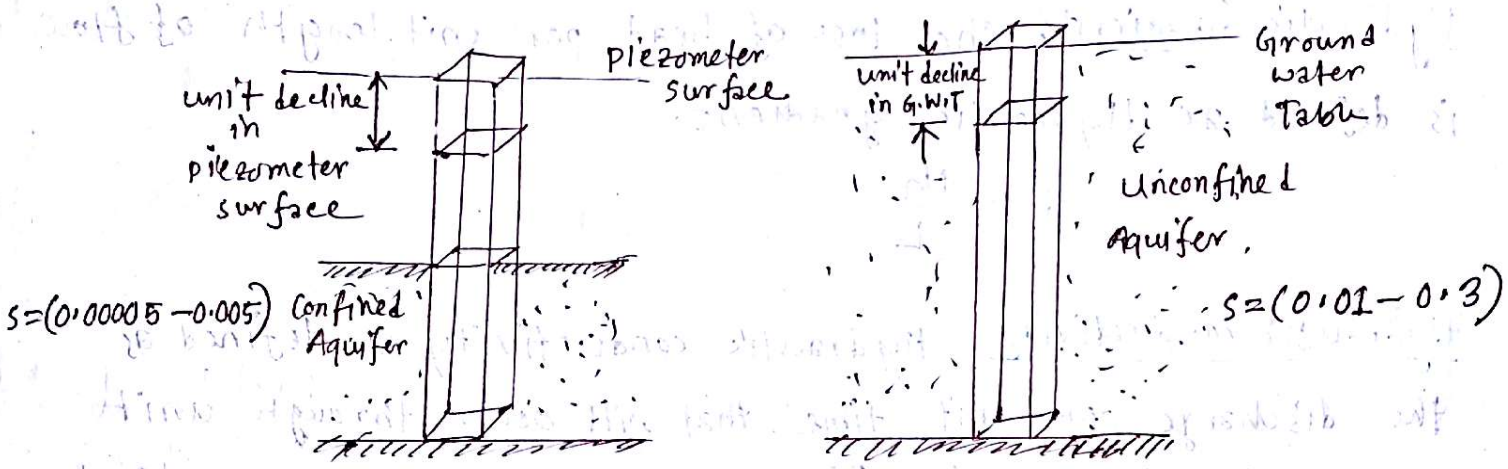

Fig. Definition sketch of storage co-efficient

co-efficient of storage is a function of the elasticity of water and aquifer skeleton. and is given by (Jacob, 1950) as.

$$S = \gamma_w b (\alpha + n\beta)$$

Where, S = Storage co-efficient, n = porosity of aquifer,

b = saturated thickness of aquifer. γ_w = unit weight of water (9810 N/m³)

$\beta = \frac{1}{K_w}$, reciprocal of bulk modulus of elasticity of water

$K_w = 2.1\ GN/m^2 = 2.1 \times 10^9\ N/m^2$

α = reciprocal of the bulk modulus of elasticity of aquifer skeleton.

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the fraction of storage attributable to expansibility water,

$S_w = 4.7 \times 10^{-6}\ nb$

Permeability: The permeability of a material is a measure of its capacity to transmit water or any other fluid through its intersection.

Hydraulic Gradient: The loss of head per unit length of flow is defined as Hydraulic gradient.

$$i = \frac{dh}{L}$$

Hydraulic conductivity: Hydraulic conductivity is defined as the discharge per unit time that will occur through unit cross sectional area of aquifer under unit hydraulic gradient.

$$K = \frac{\text{discharge}}{\text{UnitArea of aquifer}} \quad \text{(under unit hydraulic gradient)}$$

Transmissibility: Transmissibility is defined as the discharge per unit time that will occur through unit width of the aquifer under unit hydraulic gradient.

$$T = Kb$$

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Lamilar Flow:

In which flow, stream lines are parallel and flow at low velocity is called laminar flow.

The laminar flow principles are applicable for the flow of ground water in aquifer.

Darcy's Law:

It states that the rate of flow per unit area of an aquifer is proportion to the gradient of the potential head measured in the direction of flow. Darcy's law may be written as —

$$V = Ki = -K\frac{dh}{dl}$$

Assumptions: (i) The flow is laminar flow.

(ii) The flow occurs through the entire cross-section of the material without regard to solids and pores.

(iii) The soil is saturated.

Actual velocity: if V_a is the actual velocity, Then

$$Q = A \cdot V = A_p \cdot V_a \quad \text{where, } A_p = \text{area of pore space.}$$

$$\Rightarrow V_a = V \cdot \frac{A}{A_p}$$

$$= \frac{V}{(A_p/A)}$$

$$\therefore V_a = \frac{V}{n}$$

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Reynold Number: Darcy's law is found to valid for flows with Reynold's number less than 1.

The Reynold's number R_e is given by –

$$R_e = \frac{\rho v d_m}{\mu}$$

$$= \frac{v d_m}{\nu}$$

where,

d_m = dia. of grain size

v = seepage velocity

ν = Kinematic viscosity of water (m^2/sec)

Well Hydraulics

Principle objective of Ground water study:

To determine how much ground water can be safely withdrawn perennially from the aquifers in the area under study.

This determination involves –

(i) Transmissibility and storage co-efficient of the aquifers.

(ii) Leakage of the aquifer.

(iii) The effect of proposed developments on recharge and discharge conditions.

(iv) The lateral extent of each aquifer and hydraulic nature of its boundaries.

Well Hydraulics: Theory of ground water movement is well hydraulics.

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In well hydraulics, two different flow conditions are assumed:

(i) Steady flow

(ii) Non-steady flow

Steady Flow: In this type of flow, stream lines are horizontal and vertical flow components are neglected.

Assumptions:

(i) Pumping rate is constant

(ii) The well fully penetrates aquifer.

(iii) The aquifer is homogeneous, isotropic, horizontal and of infinite horizontal extent.

(iv) Water table or piezometric surface drops for the release of water from storage.

Steady Radial Flow to a well in an Unconfined aquifer:

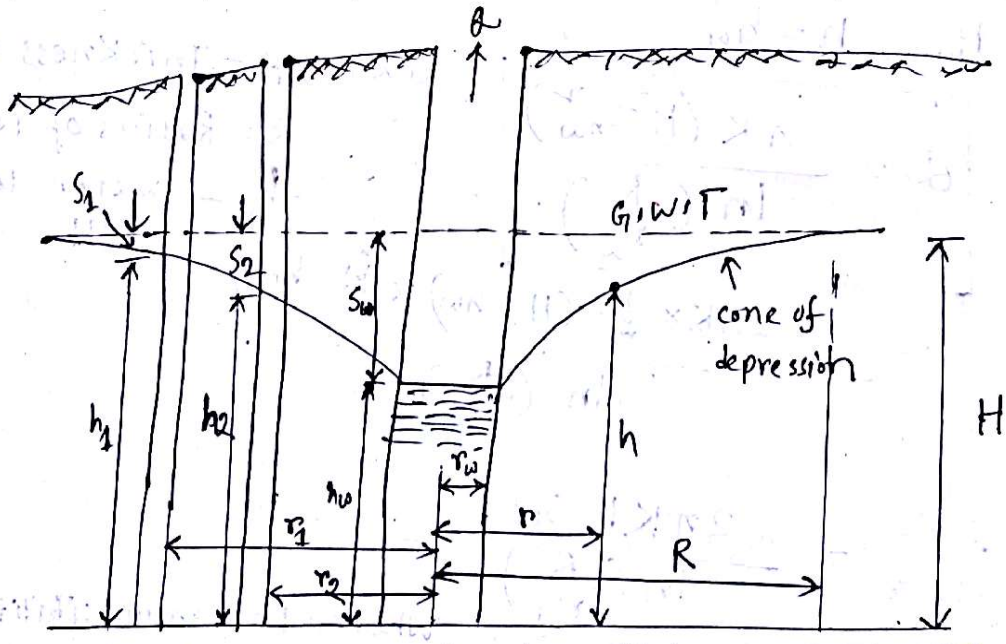

Fig. Steady flow to a well in an unconfined aquifer.

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Area of flow, $A = 2\pi rh$

$$v = ki = k \cdot \frac{dh}{dr}$$

$$Q = Av$$

$$\Rightarrow Q = 2\pi rh \times k \cdot \frac{dh}{dr} \Rightarrow h\,dh = \frac{Q}{2\pi K} \cdot \frac{dr}{r}$$

when $r = r_1$, $h = h_1$

$r = r_2$, $h = h_2$

Now,

By Integrating,

$$\int_{h_2}^{h_1} h \cdot dh = \frac{Q}{2\pi K} \times \int_{r_2}^{r_1} \frac{dr}{r}$$

$$\Rightarrow \frac{h_1^2 - h_2^2}{2} = \frac{Q}{2\pi K} \ln\left(\frac{r_1}{r_2}\right)$$

$$\Rightarrow \boxed{Q = \frac{\pi K (h_1^2 - h_2^2)}{\ln\left(\frac{r_1}{r_2}\right)}}$$

if $r = R$, $h = H$

$r = r_w$, $h = h_w$

$$\boxed{Q = \frac{\pi K (H^2 - h_w^2)}{\ln\left(\frac{R}{r_w}\right)}}$$

where, H = Thickness of aquifer
R = Radius of influence
h_w = water level in the well.

$$= \frac{2\pi K \times \frac{1}{2} \times (H + h_w) \times (H - h_w)}{\ln\left(\frac{R}{r_w}\right)}$$

$$= \frac{2\pi K b\, S_w}{\ln\left(\frac{R}{r_w}\right)}$$

$$\therefore \boxed{Q = \frac{2\pi T S_w}{\ln\left(\frac{R}{r_w}\right)}}$$

Where, T = Transmissibility of aquifer
S_w = drawdown in the well
r_w = radius of well

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Steady Radial flow to a well in a confined aquifer:

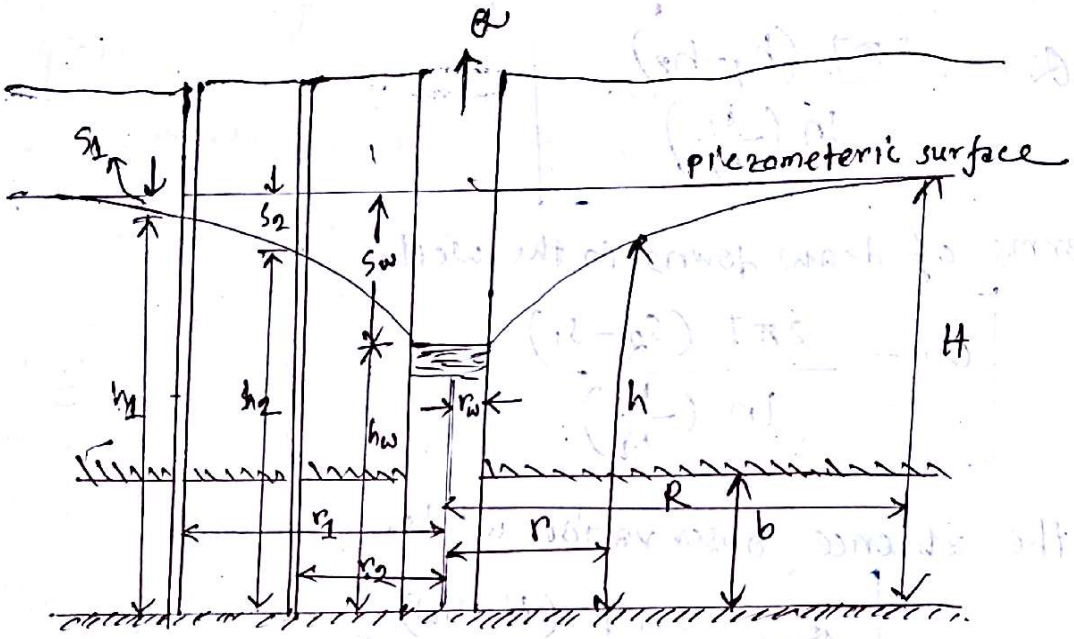

Fig. Steady flow to a well in a confined aquifer.

Aquifer thickness = b

Area of flow = $2\pi rb$

$$V = Ki = K \cdot \frac{dh}{dr}$$

$$Q = AV$$

$$\Rightarrow Q = 2\pi rb \times K \cdot \frac{dh}{dr}$$

$$\Rightarrow dh = \frac{Q}{2\pi bK} \cdot \frac{dr}{r}$$

if $r = r_1$, $h = h_1$

$r = r_2$, $h = h_2$

By integrating,

$$\int_{h_2}^{h_1} dh = \frac{Q}{2\pi Kb} \int_{r_2}^{r_1} \frac{dr}{r}$$

$$\Rightarrow h_1 - h_2 = \frac{Q}{2\pi Kb} \times \ln\left(\frac{r_1}{r_2}\right)$$

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$$\Rightarrow h_1 - h_2 = \frac{Q}{2\pi T} \ln\left(\frac{r_1}{r_2}\right)$$

$$\Rightarrow \boxed{Q = \frac{2\pi T (h_1 - h_2)}{\ln\left(\frac{r_1}{r_2}\right)}}$$

In terms of draw downs in the well,

$$\boxed{Q = \frac{2\pi T (s_2 - s_1)}{\ln\left(\frac{r_1}{r_2}\right)}}$$

In the absence observation wells,

$$\boxed{Q = \frac{2\pi T (H - h_w)}{\ln\left(\frac{R}{r_w}\right)}}$$

$$\boxed{Q = \frac{2\pi T s_w}{\ln\left(\frac{R}{r_w}\right)}}$$

Unsteady flow: The position of the piezometer surface near the well changes continuously with time.

Assumptions:

(i) The aquifer is confined

(ii) The well diameter is very small

(iii) The aquifer is dewatered instantaneously as the piezometric head drops.

The equation is given by -

$$s = \frac{Q}{4\pi T} \int_u^\infty \frac{e^{-u}}{u} \, du$$

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where, s = drawdown in an observation well at a distance r from pumping well after 't' seconds since pumping began.

Q = constant pumping rate from the well

T = Transmissibility of the aquifer.

and,

$u = \frac{r^2 s}{4Tt}$ Here, s = storage co-efficient of aquifer.

$$\Rightarrow s = \frac{Q}{4\pi T} W(u)$$

well function or Theis well function,

$$W(u) = \left[-0.577216 - \ln u + u - \frac{u^2}{2 \cdot 2!} + \frac{u^3}{3 \cdot 3!} \quad \frac{u^4}{4 \cdot 4!} + \cdots\right]$$

Theis Method:

$$u = \frac{r^2 s}{4Tt}$$

$$\Rightarrow \frac{r^2}{t} = \left(\frac{4T}{s}\right) \times u$$

and, $s = \left(\frac{Q}{4\pi T}\right) W(u)$

comparing these two equation, $s \leftrightarrow \frac{r^2}{t}$ and $W(u) \leftrightarrow u$

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Theis suggested a graphical procedure to obain an approximate solution of S and T. The procedure is a follows:

1. $\frac{r^2}{t}$ vs. S in transparent logarithmic paper. (Data Curve)

2. W(u) vs. U in logarithmic paper. (Type curve)

3. Data curve is superimposed on the type curve. (Matching)

4. Then, compute the value T and S. $\boxed{S = \frac{4Tu}{(r^2/t)}}$ and $\boxed{T = \frac{Q}{4\pi s} W(u)}$

Cooper and Jacob Method:

$$S = \frac{Q}{4\pi T} W(u) \quad \text{and} \quad W(u) = \left[-0.5772 - \ln u + u - \frac{u^2}{2.2!} + \cdots\right]$$

For two observation well,

$$S_1 = \frac{Q}{4\pi T} W(u_1) \quad \text{where, } u_1 = \frac{r^2 S}{4Tt_1}$$

$$\text{and } S_2 = \frac{Q}{4\pi T} W(u_2) \quad ; \quad u_2 = \frac{r^2 S}{4Tt_2}$$

$$\Delta S = S_2 - S_1$$

$$= \frac{Q}{4\pi T} \left[W(u_2) - W(u_1)\right]$$

Since, u_1 and u_2 are small,

$$\Delta S = \frac{Q}{4\pi T} \left[(-0.5772 - \ln u_2) - (-0.5772 - \ln u_1)\right]$$

$$= \frac{Q}{4\pi T} (\ln u_1 - \ln u_2)$$

$$= \frac{Q}{4\pi T} \ln\left(\frac{u_1}{u_2}\right)$$

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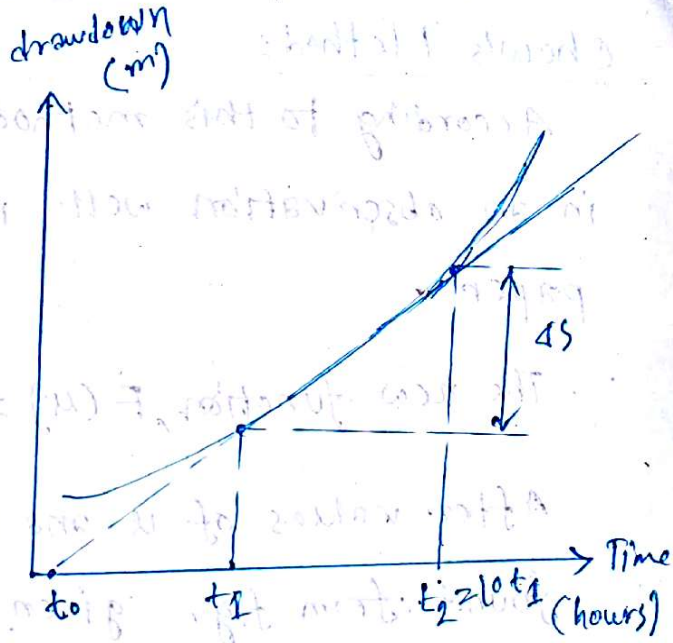
$$\Rightarrow \Delta s = \frac{Q}{4\pi T} \ln\left(\frac{t_2}{t_1}\right)$$

$$\therefore T = \frac{Q}{4\pi \Delta s} \ln\left(\frac{t_2}{t_1}\right)$$

$$= \frac{2.3026\, Q}{4\pi \Delta s} \log\left(\frac{t_2}{t_1}\right)$$



Now, since $t_2 = 10\, t_1$

$$\boxed{T = \frac{2.3026\, Q}{4\pi \Delta s}} \quad \text{..... (i)}$$

To evaluate s, following procedure is adopted.

$$s = \frac{Q}{4\pi T} W(u)$$

when, $s = 0$, $W(u) = 0$

$$-0.5772 - \ln u = 0$$

$$\Rightarrow u = 0.56146 = \frac{S r^2}{4 T t_0}$$

$$\Rightarrow \boxed{S = \frac{2.246\, T t_0}{r^2}} \quad \text{..... (ii)}$$

Equation (i) & (ii) are used to evaluate T and S after obtaining Δs and t_0 from time-drawdown curve.

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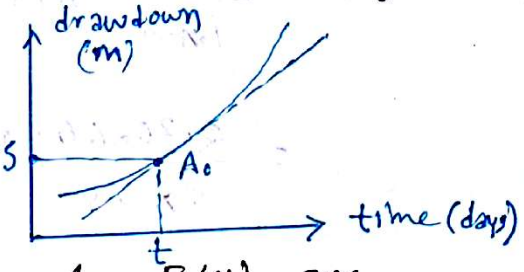
Chow's Method:

According to this method, the data on time-draw down in an observation well is plotted on ~~curve~~ a semi logarithic paper.

The new function, $F(u) = \frac{S}{4\pi}$



After values of u and W(u) corresponding to F(u) are found from fig. given by chow.

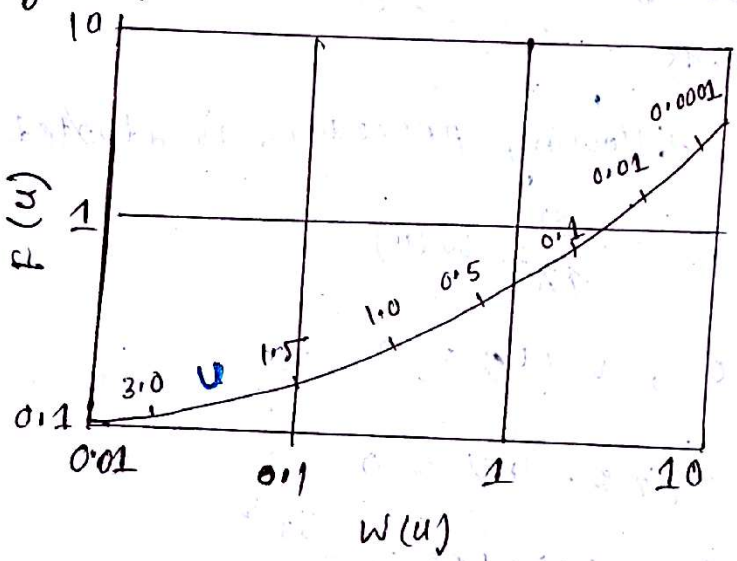


Fig. Relation between F(u), W(u) and u for Chow's Method

Since the relation between F(u), W(u) and u is given by,

$$F(u) = \frac{W(u)\, e^{u}}{2.3}$$

After knowing u and W(u), the aquifer parameter T and S are finally computed using these two equation:

$$T = \frac{Q}{4\pi s} W(u) \quad \text{and,} \quad S = \frac{4 T \cdot u \cdot t}{r^2}$$

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Cone of Depression:

If water is pumped at a constant rate from the well, a gradient in the water table towards the well is created which results in a depressing form of the water table. This is called the cone of depression.

Drawdown:

The decrease in water level at the well is called drawdown. If the drawdown is small compared to the thickness of the aquifer, the streamlines of the flow to well may be assumed to be horizontal.

Difference between Aquifer, Aquiclude, Aquitard and Aquifuge.

Aquifer	Aquiclude	Aquitard	Aqufuge
1. High porosity	1. High porosity	1. Medium porosity	1. less porosity.
2. Large amount of water	2. large amount of water	2. Appreciable quantities of water	2. No water
3. permeable	3. impermeable or less permeable	3. poorly permeable	3. impermeable
4. can transmit water	4. can not transmit water.	4. does not yield water freely to well, but may transmit	4. can not transmit water.
5. Example: sandy layer	5. Example: clay layer	5. Example: Sandy clay layer.	5. Example: Solid granite Rock

Scanned by CamScanner

Seepage velocity:

Seepage velocity is the apparent velocity through bulk of the porous medum.

It is not the actual velocity of water in the pores. It is calculated from Darcy's law. $v_s = ki$

Actual velocity:

Actual velocity is the velocity of water in the soil pores. It is higher than sepage velocity.

$$v_a = \frac{v_s}{n}$$

Scanned by CamScanner

GROUND WATER RECHARGE

Groundwater Recharge:

1)Developing of artificial underground reservoir by artificial recharging for storing water underground called recharging of underground water.

2)It is quite advantages as compared with dams,reservoiers etc.

3)Artificial recharging technique is under intensive research and is being increasingly used in france, Germany etc.

What is Artificial Recharge

- Artificial recharge is the process by which the ground water recharge is augmented at the rate much higher then those under natural condition of percolation.

Why Artificial Recharge

- In most low rainfall areas of the country the availability of utilizable surface water is so low that people have to depend largely on ground water for agriculture and domestic uses.
- So in order to improve the ground water situation it is necessary to artificially recharge the depleted ground water aquifer.

Advantage of artificial recharge

- To enhance the groundwater yield in depleted the aquifer due to urbanization .
- Conservation and storage of excess surface water for future requirements.
- To improve the quality of existing groundwater through dilution
- To improve bacteriological and other impurities from sewage and waste water by natural filtration , so that water is suitable for re use .

TECHNIQUES

Direct Methods

Surface method: to enhance groundwater infiltration by providing more residence time with the help of structural and non-structural measures

Subsurface method

Indirect Methods

Induced Recharge Method

Aquifer Modification Method

SURFACE METHODS

SUBSURFACE METHODS

Percolation tanks

Fig. A percolation tank about to get dry towards beginning of summer. Location: Village: Ralegan Siddhi, District: Nagar, Maharashtra State.

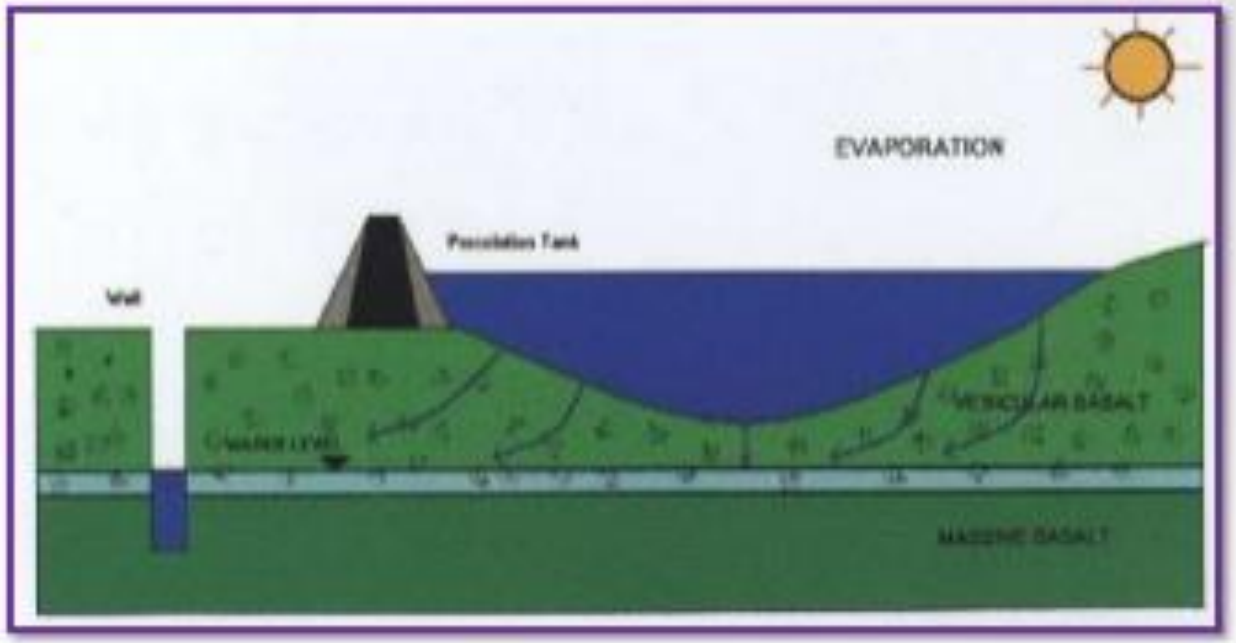
Source: www.indiawaterportal.org

- This is the most common method of artificial recharge.
- In this method , water is impounded in series of basins or percolation tank ,
- The size of basin may depend upon the topography of area, in flatter area will have large basin .
- This method is applicable in alluvial area as well as hard formation .
- The efficiency and feasibility of this method is more in hard rock formation where the rocks are highly fractured and weathered

Percolation Tank

PERCOLATION TANK

- Series of earthen dams are constructed on suitable sites for storing of adequate quantity of surface water.
- Tank area should be selected in such a way that significant amount of water infiltrates through the bed of the tank and reaches the groundwater table.



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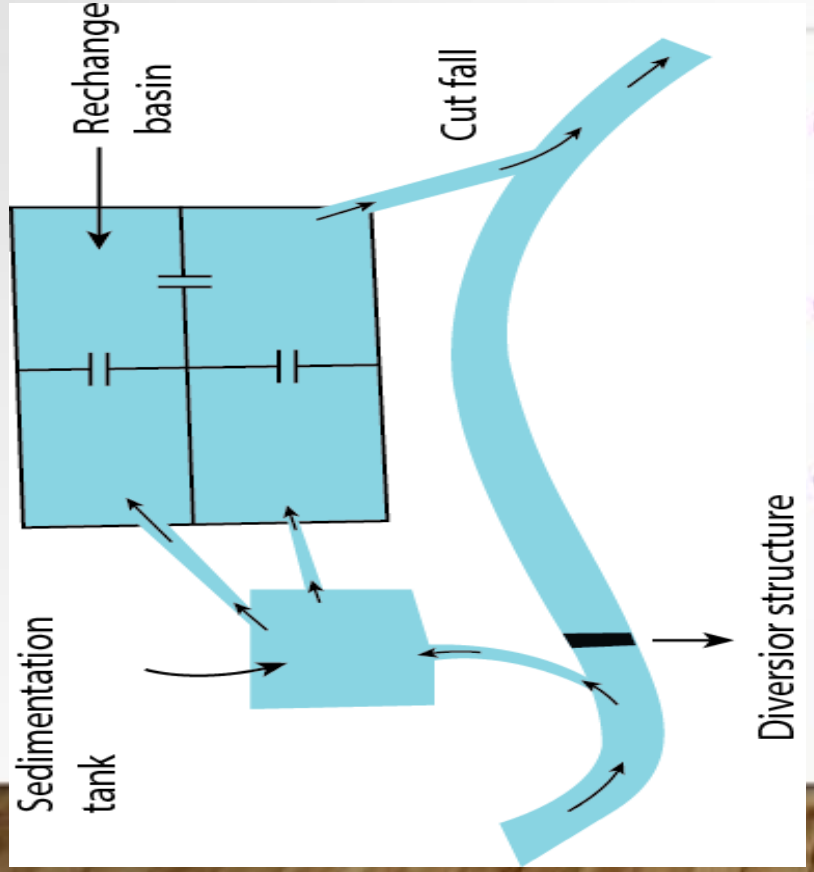
FLOODING



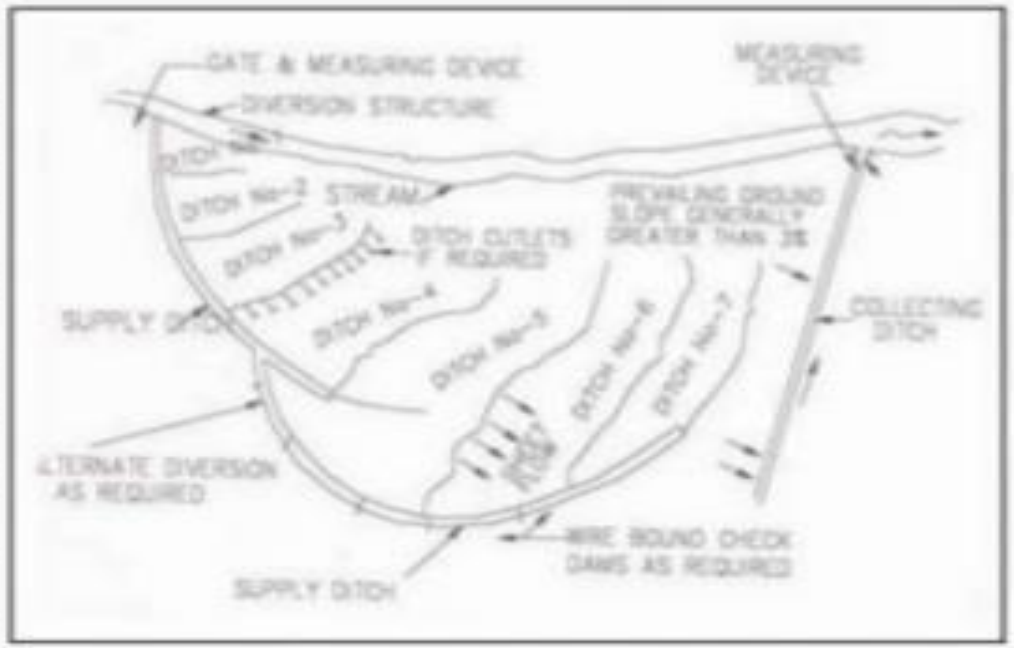
- Flat region where water can be spread as a thin layer.
- Water is distributed over the region using a distribution system.
- This method can achieve higher rate of infiltration in a region having thin vegetation cover or sand soil cover.

Stream Augmentation

Source: www.indiawaterportal.

- Seepage from natural stream or rivers is one of the most important source of recharge of the ground water reservoir.
- When total water supply available in the stream /river exceeds the rate of infiltration ,the excess is lost as runoff .
- This runoff can be arrested through check bunds or widening the steam beds thus larger area is available to spread the river water increasing the infiltration .
- The site selected for check dam should have sufficient thickness of permeable bed or weathered formation to facilitate recharge of stored water with in short span of time.
- The water stored in these structures is mostly confined to stream course and height is normally less than 2m. To harness maximum runoff, a series of such check dam may be constructed.

Ditch & Furrow System

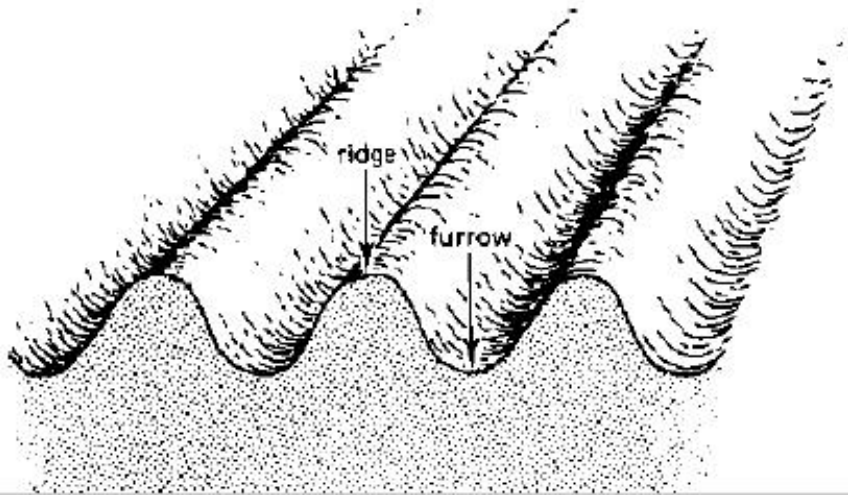


Source: megphed.gov.in

- In areas with irregular topography ditches or furrow provide maximum water contact area for recharge .
- This technique consists of a system of shallow flat bottomed and closely spaced ditches/ furrow which are used to carry water from source like stream/canals and provide more percolation opportunity.
- This technique required less soil preparation less is less sensitive to silting.

Furrow System

Furrow irrigation by opening the bank or dyke of the ditch



Contour bund

Contour bund is a small embankment constructed along the contour in hilly region to retain the surface runoff for longer time. This scheme is adopted for low rainfall area where internal subsurface drainage is good.

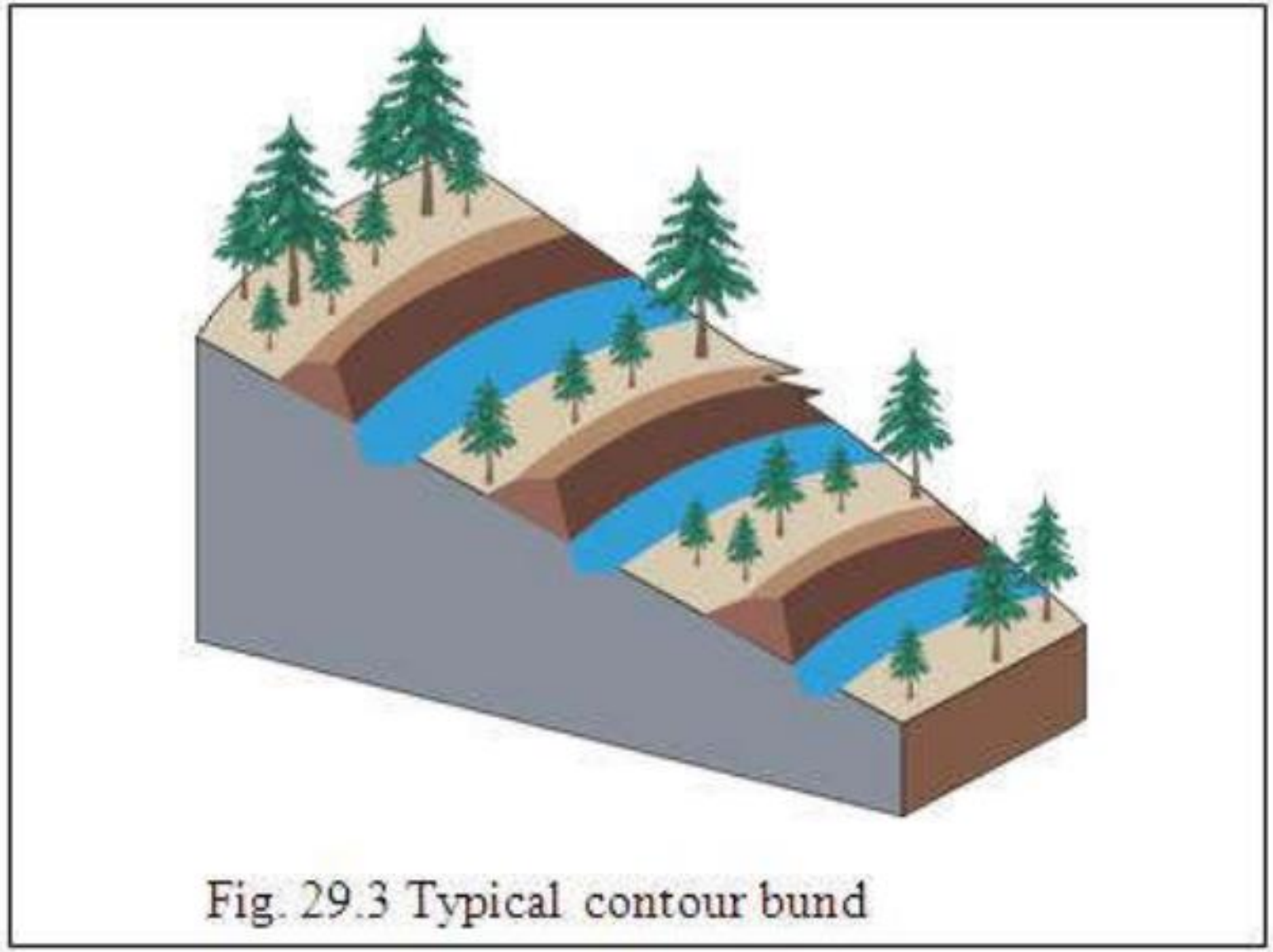
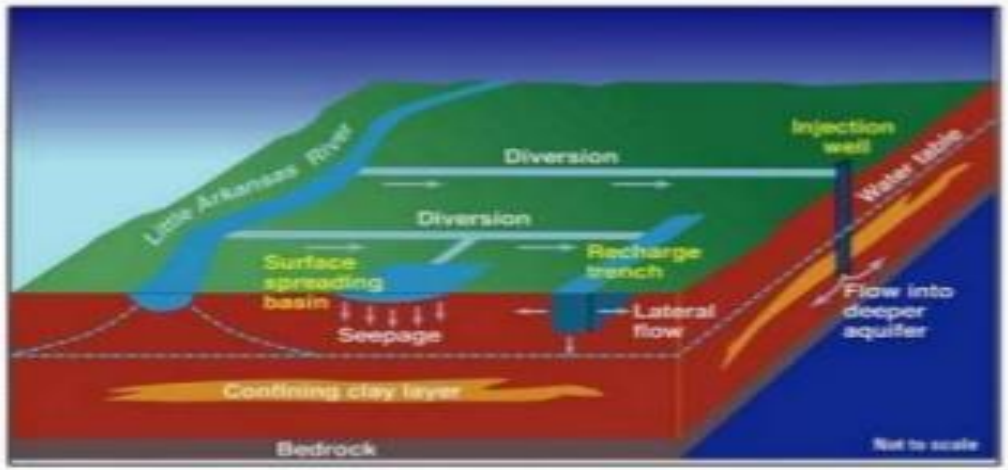


Fig. 29.3 Typical contour bund

B.Sub – Surface Method

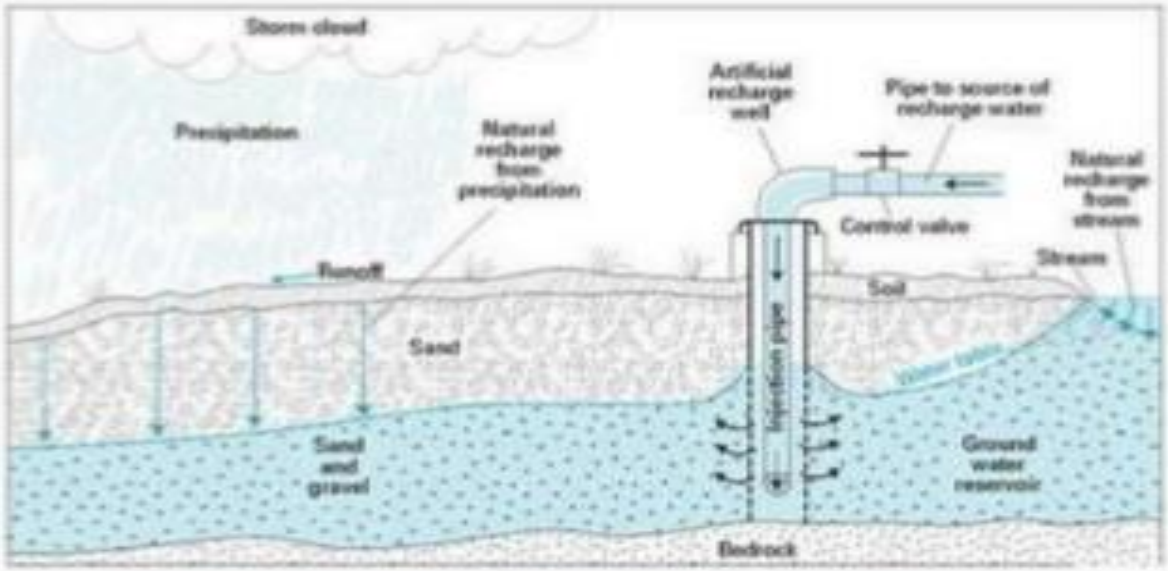


Source: www.indiawaterportal.org

Figure 5 : sub surface method

- In this method the structure lies below the surface and recharges ground water directly.
- The important structures commonly use are recharge wells, recharge shaft, dug wells etc

Recharge well

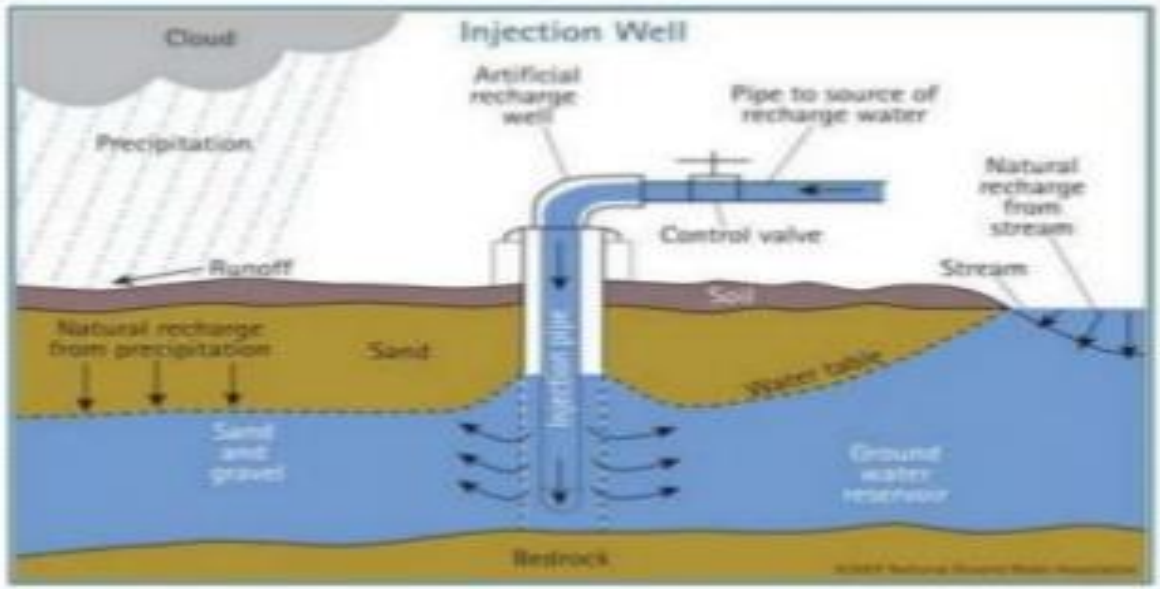


Source:www.ngwa.org

Figure 6 : Injection well

- Recharge wells can be of two types-
- (a) Injection well, where water is " pumped in" for recharge and
- (b) Recharge well, where water flows under gravity.

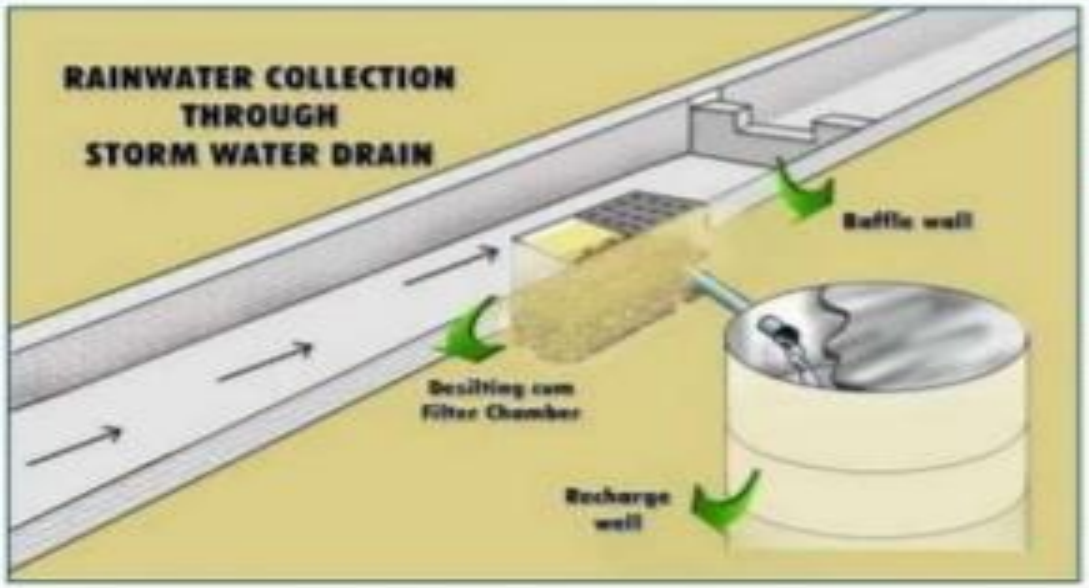
(a) Injection Well



Source: www.ngwa.org

- The injection wells are similar to a tube well.
- This technique is suitable for augmenting the ground water storage of deeper aquifers by "pumping in" treated surface water.
- These wells can be used as pumping wells during summers.
- The method is suitable to recharge single aquifer or multiple aquifers.
- The recharge through this technique is comparatively costlier and required specialized technique.

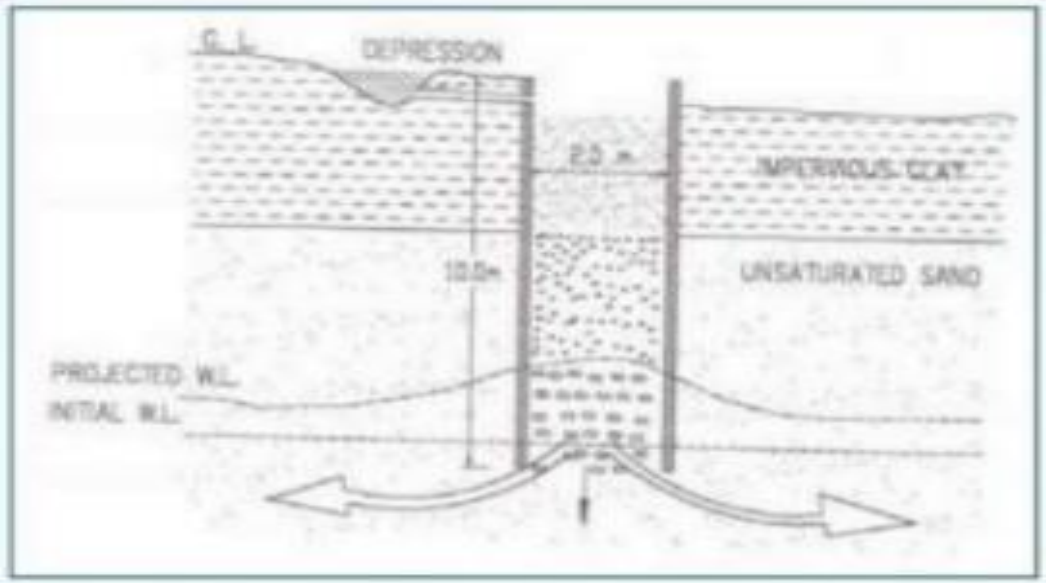
Recharge Well



Source: www.indiawaterportal.org

- The recharge well for shallow water table aquifers up to 50 m are cost effective because recharge can take place under gravity flow only.

2.Pitch & Shafts

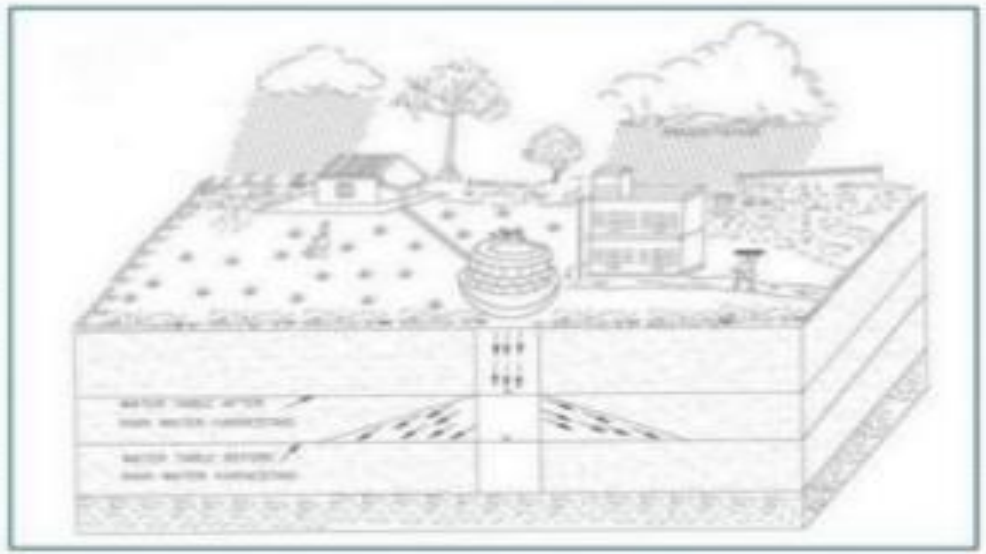


Source: megphed.gov.in

Vertical recharge shaft

- In area where impervious layer is encountered at shallow depth the pits & shafts are suitable structure for artificial recharge.
- These structures are cost effective to recharge the aquifer directly.
- The diameter of shaft should normally be more than 2m to accommodate more water.
- The advantage of shafts/ pits structures is that they do not requires large pieces of land like percolation tank & other spreading method.
- There are practically no losses of water in form of soil moisture and evaporation like other methods of spreading.

Dug Wells



Source: megphed.gov.in

Recharge through Dug Wells

- In alluvial as well as hard rock areas there are thousand of dug wells have either gone dry due to considerable decline of water levels.
- These dug wells can be used as recharge structure storm water and other surplus water from canal etc. can be diverted into these structure to directly recharge the dried aquifer.
- The water for recharge should be guided through a pipes to the bottom of well to avoid entrapment of bubbles in the aquifer.

LECTURE ON

DESIGN OF WATER WELLS

CE 3221, Hydrology, Credit=3.00, Contact Hours=3/week

Introduction:

A water well has to be designed to get the optimum quantity of water economically from a given geological formation

- The water requirements for the particular schemes –rural water supply , agricultural and industrial needs, has to be carefully determined

- The choice of open wells or bore wells (tube wells) and the method of well design depends upon-

-Topography

-Geological conditions of the underlying strata

-Depth of GW table

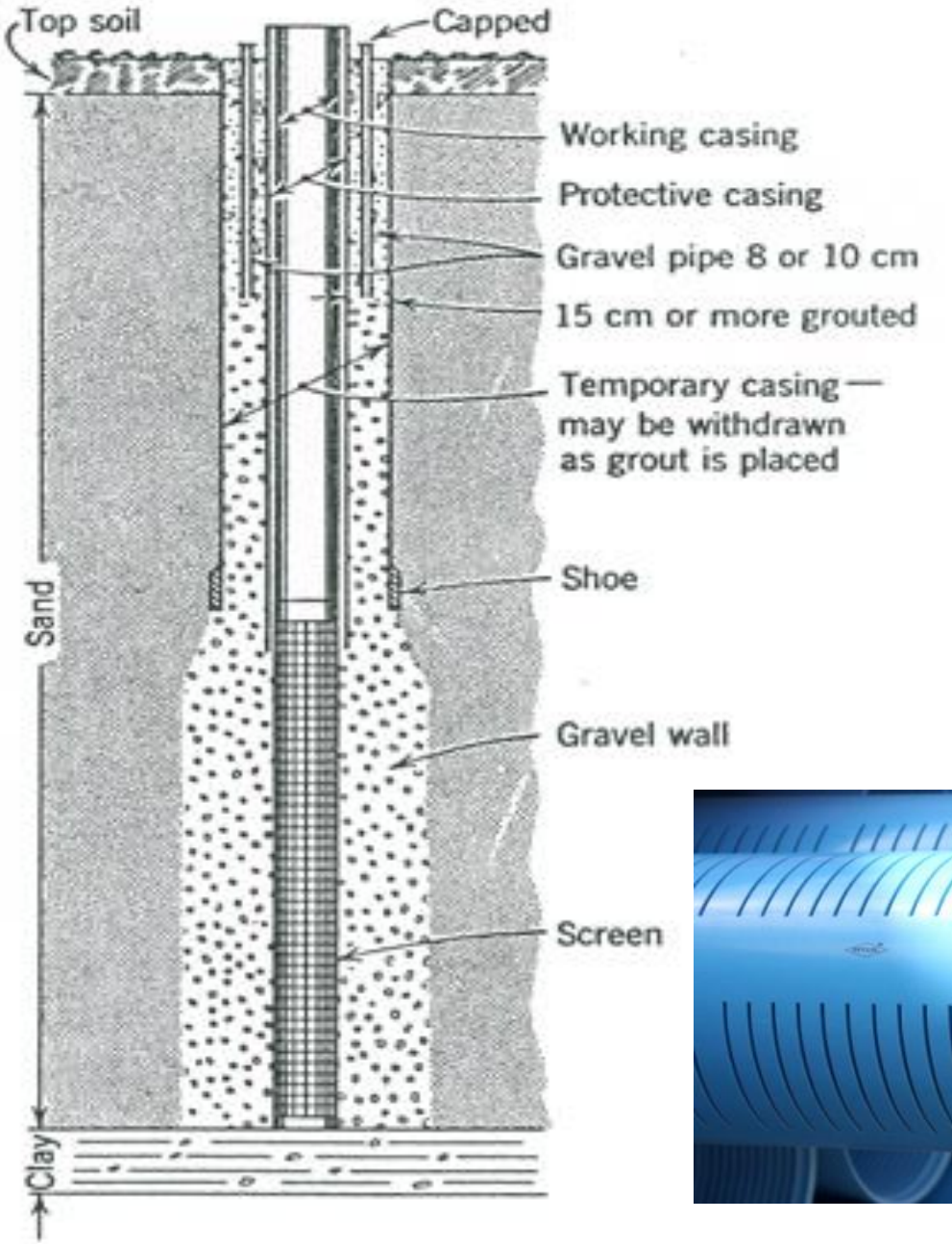
-Rainfall

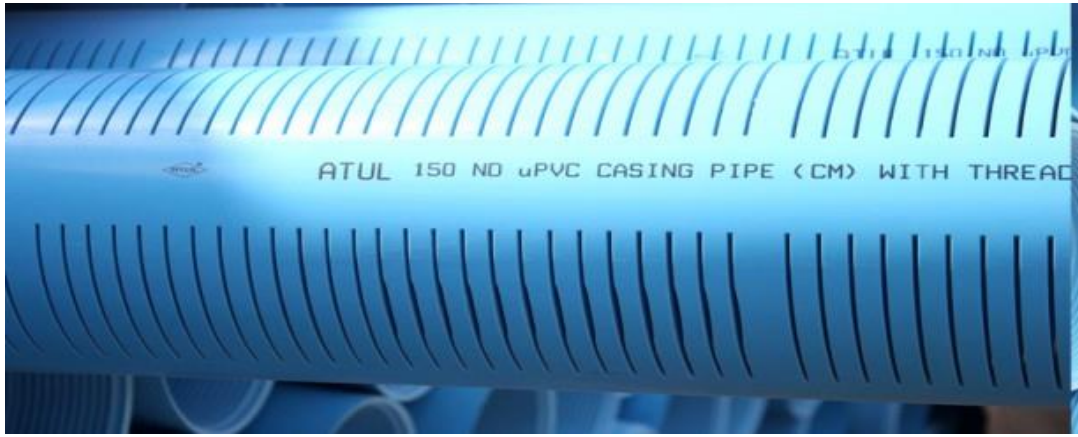
-Climate

-The quantity of water required

Introduction Cont…

- A water well design involves selection of proper dimensions like the diameter of the well and that of the casing, length and the location of the screen including slot size, shape and percentage of opening area


Top soil
Capped
Working casing
Protective casing
Gravel pipe 8 or 10 cm
15 cm or more grouted
Temporary casing — may be withdrawn as grout is placed
Shoe
Sand
Gravel wall
Screen
Clay


ATUL 150 ND uPVC CASING PIPE (CM) WITH THREAD

Well Diameter

The Size of the well diameter should be properly chosen since it significantly affects the cost of well construction.

The diameter must be chosen to give the desired percentage of open area in the screen (15 to 18%), so that the entrance velocities near the screen do not exceed 3 to 6 cm/sec, so as to reduce the well losses and hence the drawdown, to exclude the finest particles of sand from migrating near the slots and prevent incrustation and corrosion at the strainer slots.

Well Diameter

Depuit's equation

$$Q\alpha \frac{1}{log10\frac{R}{r_w}}$$

For R=300 m, a 60 cm well yields only 25% more than a 15 cm well and 12% more than a 30 cm well, which shows that Drilling a large diameter well will not necessarily mean proportionally large yields

Well Depth

- The depth of a well and the number of aquifers it has to penetrate is usually determined from the lithological log of the area.

- An experienced driller can decide the depth at which drilling can be stopped after being advised by the hydrologist who analyses the samples collected during the drilling.

- The well is usually drilled up to bottom of the aquifer so that the full aquifer thickness is available, permitting greater well yield.

Design of well Screen

- Screen Length: In homogeneous artesian aquifer about 70 to 80% (3/4)of the aquifer thickness is screened.

- In case the non-homogeneous artesian aquifer, it is best to screen the most Permeable strata.

- Theory and experience have shown that screening the bottom one- third of the aquifer provides the optimum design.

Design of slot Size

- The size of slots depends upon the gradation and size of the formation material

- In case of naturally developed wells the slot size is taken as 40 to 70% of the size of formation materials.

- If the slot size selected on this basis becomes smaller than 0.75mm, then It calls for an artificial gravel pack.

- Artificial gravel pack is required when the aquifer material is homogeneous with Uniformity co-efficient less than 3.00 and effective grain Size less than 0.25 mm

Screen Diameter

After the length of the screen (depending upon the aquifer thickness) and the slot size (based on the size and gradation of the aquifer materials) have been selected , the screen diameter is determined so that the **entrance velocities near the Screen will not exceed 3 to 6 cm/sec to prevent incrustation and corrosion and to minimize friction losses**

Selection of Screen

Selection of screen material depends on-

-mineral content of water

-presence of bacterial slimes

-strength requirements

Selection of Screen

The Screen material should be resistant to incrustation and corrosion and should have strength to withstand the column load and collapse pressure.

The **principle** indicators of corrosive ground water are
-low pH
-Presence of dissolved oxygen
-CO_2> 50ppm (parts per million or mg/l)
-CL> 500ppm

The **principal indicators of incrustating ground water** are
-total Hardness> 330 ppm
-total alkalinity> 300ppm
-iron content> 2 ppm
-pH> 8

Open wells Versus Borewells

In choosing the type of well the following factors have to be considered

1. Availability of space
2. **Hydrological characteristics** of the subsurface strata
3. Seasonal **fluctuation** of water levels
4. **Cost** of well construction including provision of water lifting appliances
5. **Economics and ease** of water lifting operation

Open wells -Advantages

1. **Storage capacity** of water is available in the well itself

2. Do not require sophisticated equipment and skilled personnel for construction

3. **Can be easily operated** by installing a centrifugal pump at different Settings for low and high water levels

Open wells -Disadvantages

1.Large space is required for the well and for excavated material lying on the surface like a big mound
2.Construction is slow and laborious
3.Subject to high fluctuations of water table during different seasons
4. Susceptibility to dry up in years of drought
5. High cost of construction as the depth increases in hard rock areas
6. Deep seated aquifer cannot be economically tapped
7. Uncertainty of tapping water of good quality
8. **Susceptibility for contamination or pollution** unless sealed from surface water ingress

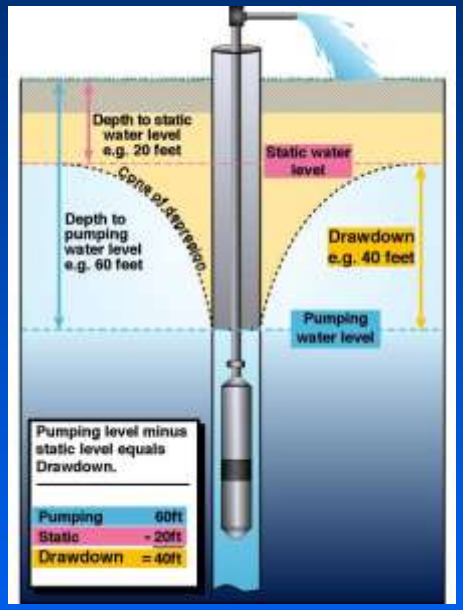
Tube (bore) wells -Advantages

1. Do not require much space
2. Can be constructed quickly
3. Fairly sustained yield of water can be obtained even in years of scanty rainfall
4. Economic when deep –seated aquifers are encountered
5. Generally good quality of water is tapped

Tube (bore) wells -Disadvantages

1.Require costly and complicated drilling equipment and machinery
2.Required skilled workers and great care to drill and complete the tube wells
3.Installation of costly turbine or submersible pumps is required
4. Possibility of missing the fracture , fissures and joints in hard rock areas resulting in many dry holes

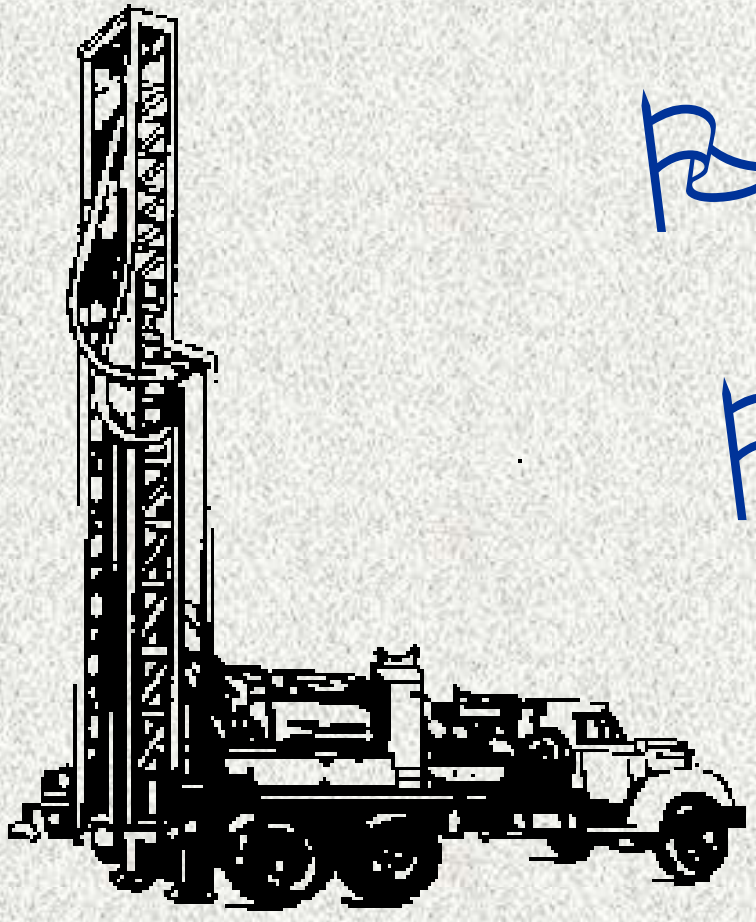
WATER WELL
Drilling & Construction



WATER WELL DESIGN

- Provide well that meets needs of owner
- Provide suitable quality water (potable and turbidity-free for drinking water wells)
- Provide long service life (25+ years)

Types of Water Wells



- DRILLED
- DRIVEN
- DUG

DRILLED WELLS

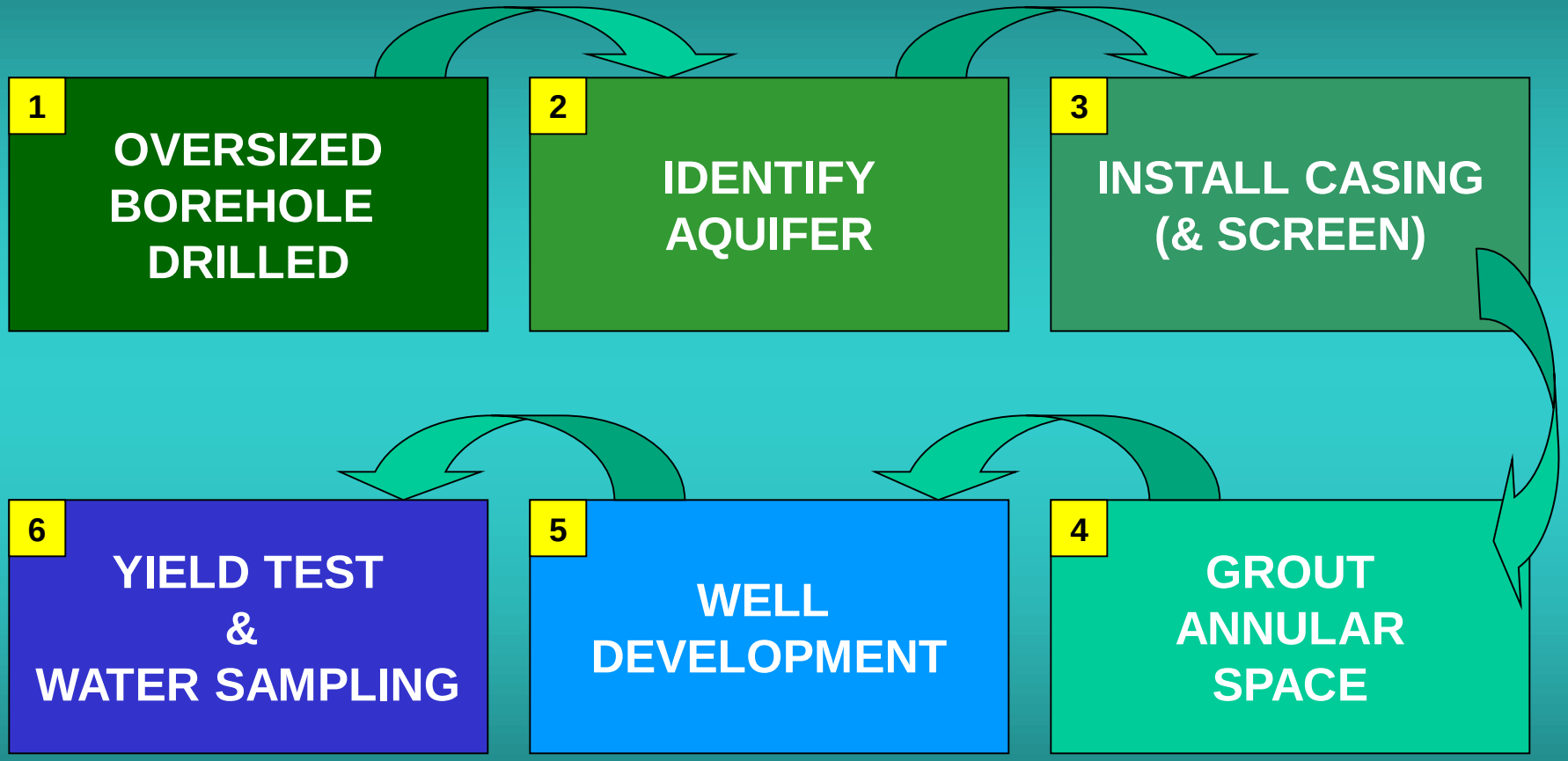
➔ Terminated in glacial drift (sand, gravel) or bedrock

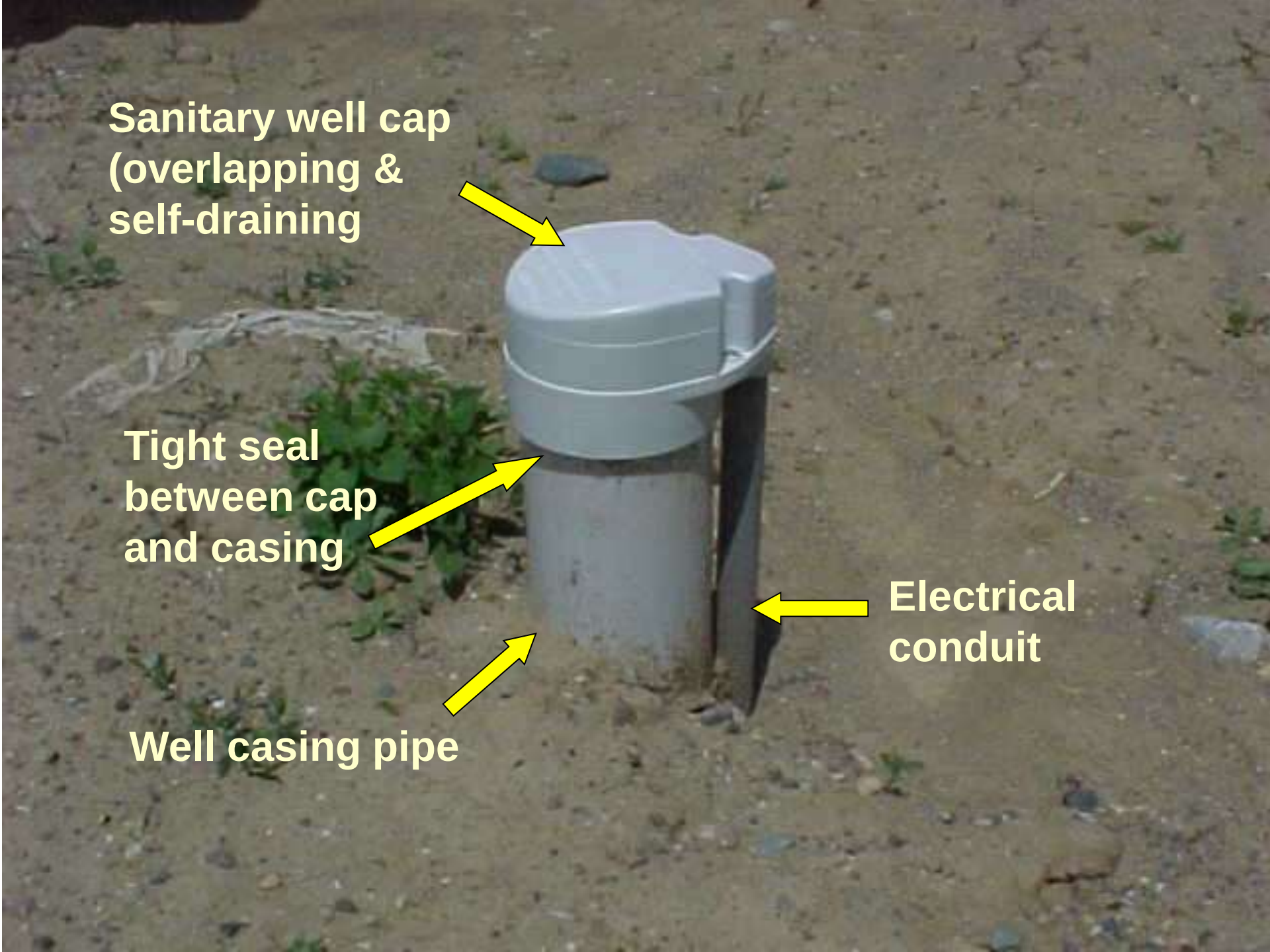
➔ Constructed with rotary, cable tool, jetting, hollow rod or auger drilling methods

Rotary

Cable Tool

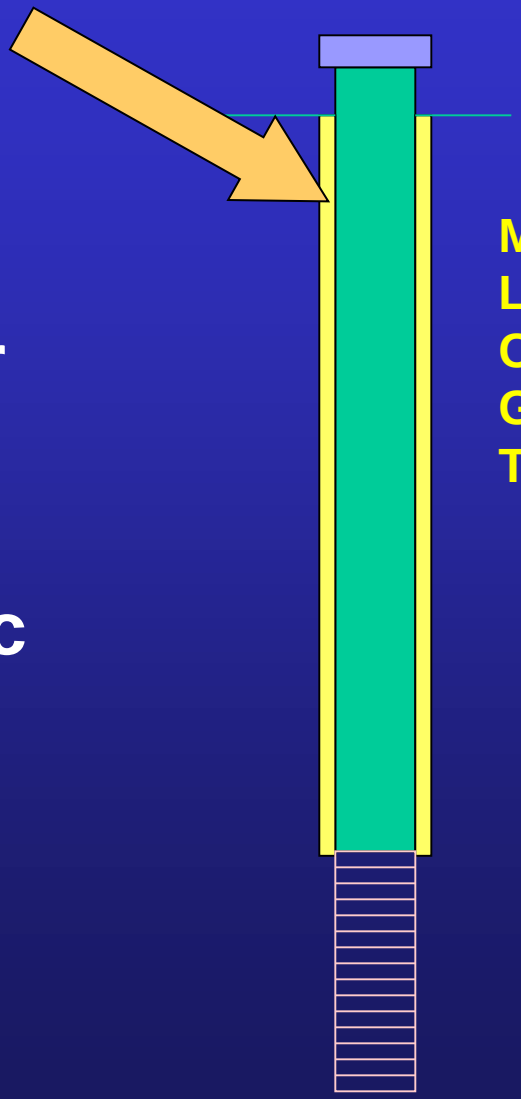
TYPICAL ROTARY WELL CONSTRUCTION SEQUENCE




Sanitary well cap (overlapping & self-draining
Tight seal between cap and casing
Electrical conduit
Well casing pipe

BOREHOLE

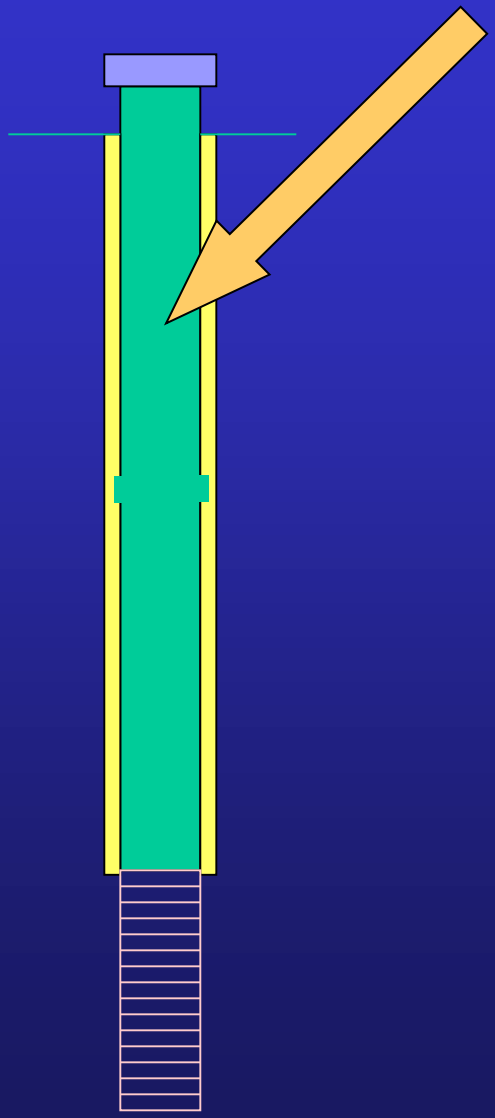
Vertical circular boring to reach aquifer (water bearing geologic material)



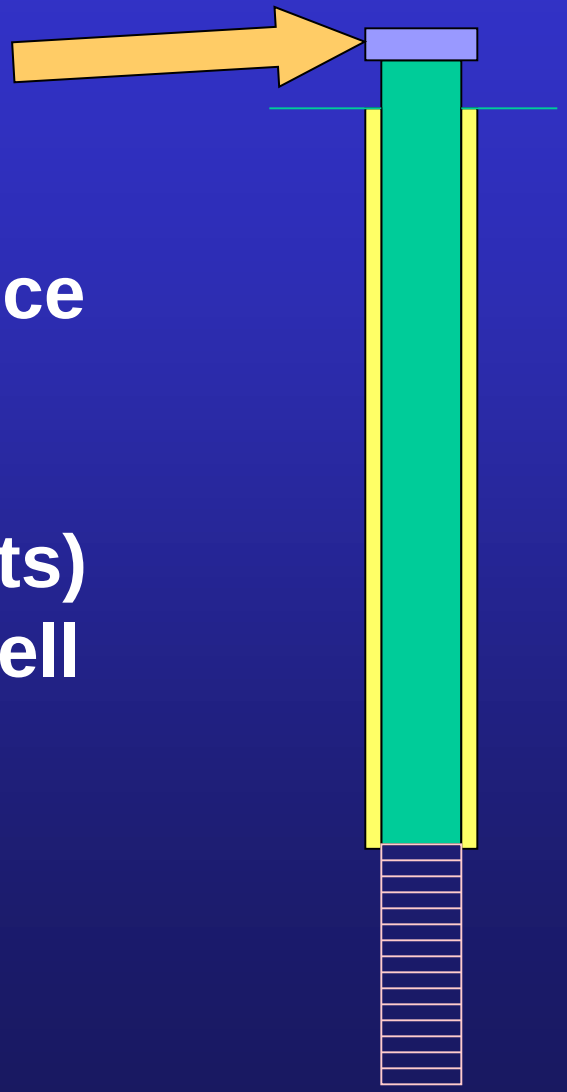
MINIMUM 2 IN. LARGER THAN CASING IF GROUTING THRU CASING

CASING

Steel or plastic pipe installed to keep borehole wall from collapsing



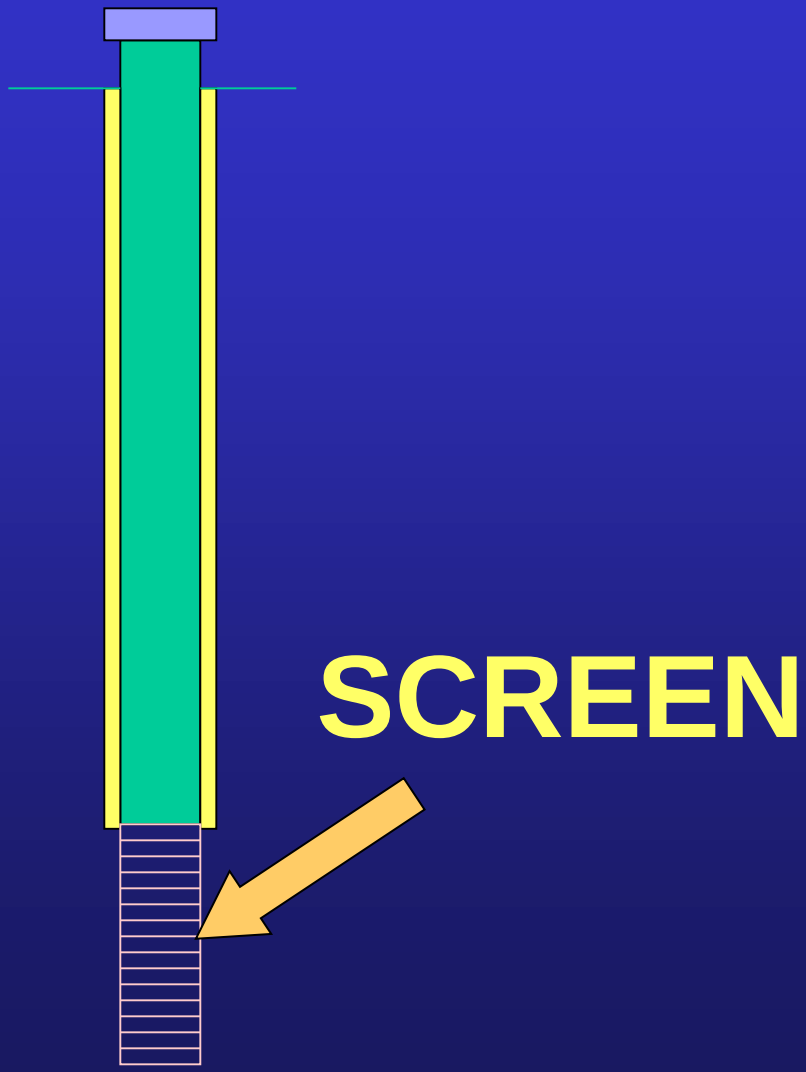
WELL CAP or SEAL



Mechanical device to prevent contaminants (including insects) from entering well casing

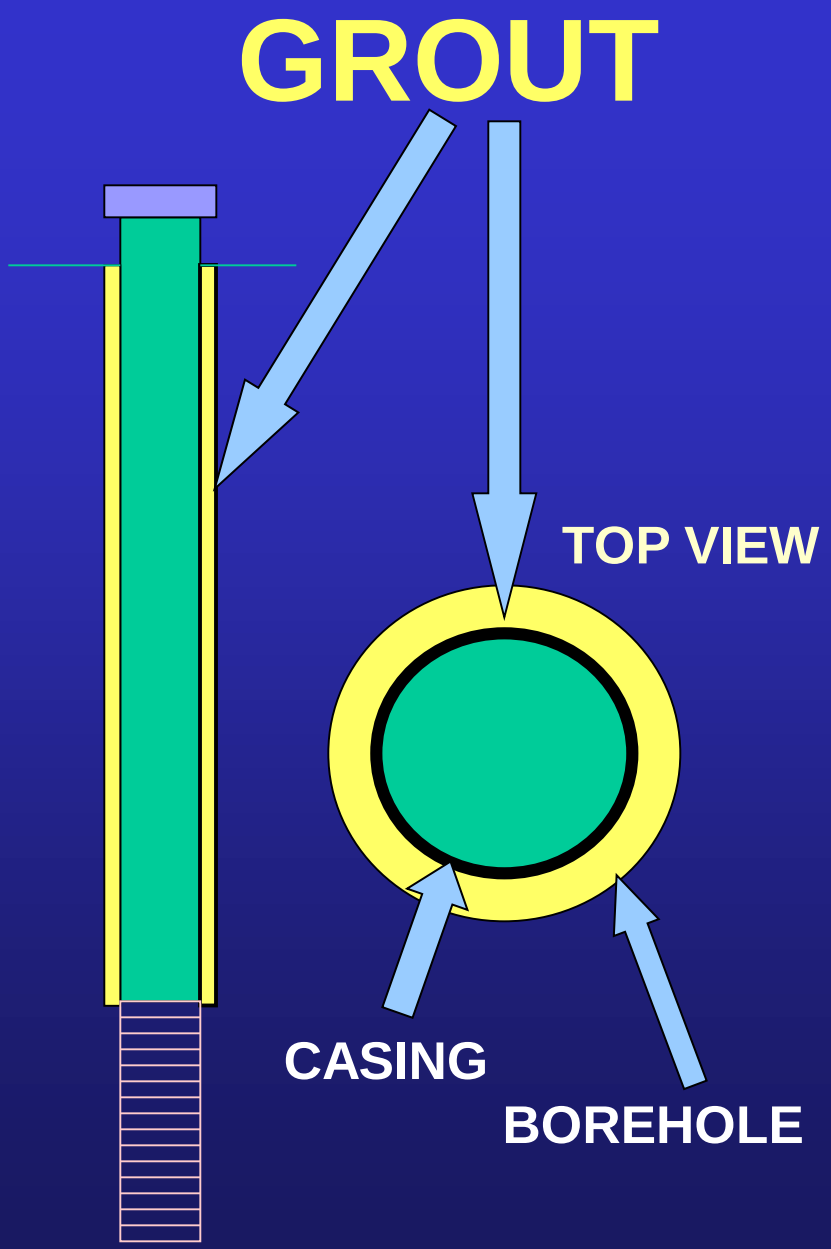
Intake device to allow water to enter well and keep sand out

Wire-wrapped screen most common



Impermeable cement or bentonite clay slurry placed in annular space between borehole and casing to:

- prevent well contamination



WELL GROUTING MATERIALS

TYPE	*COMPOSITION*	*CHARACTERISTICS*
BENTONITE SLURRY	POWDERED BENTONITE & WATER GRANULAR BENTONITE, POLYMER & WATER	▪FLEXIBLE LOWER STRENGTH SEAL ▪MOST POPULAR DUE TO LOWER COST AND TARGETED MARKETING ▪WASH-OUT UNDER ARTESIAN PRESSURE ▪NO HEAT OF HYDRATION
NEAT CEMENT	PORTLAND CEMENT & WATER	▪MORE WIDELY USED IN OIL FIELD THAN WATER WELLS ▪HIGHER STRENGTH RIGID SEAL ▪BEST CHOICE FOR BEDROCK WELLS & FLOWING WELLS ▪HEAT OF HYDRATION & MICROANNULUS CONCERNS
CONCRETE GROUT	PORTLAND CEMENT, SAND & WATER	▪MORE PERMEABLE THAN NEAT CEMENT GROUT ▪MORE DIFFICULT TO PUMP (ABRASIVE) ▪GOOD CHOICE FOR LARGE DIAMETER WELLS

CASING MATERIALS COMPARISON

PVC PLASTIC	vs.	STEEL
Non-corroding		Corrodes
Lower strength		Higher strength
Fewer water quality complaints		Rusty water
Rotary construction only		Suitable for any drilling method
1/3 cost of steel		

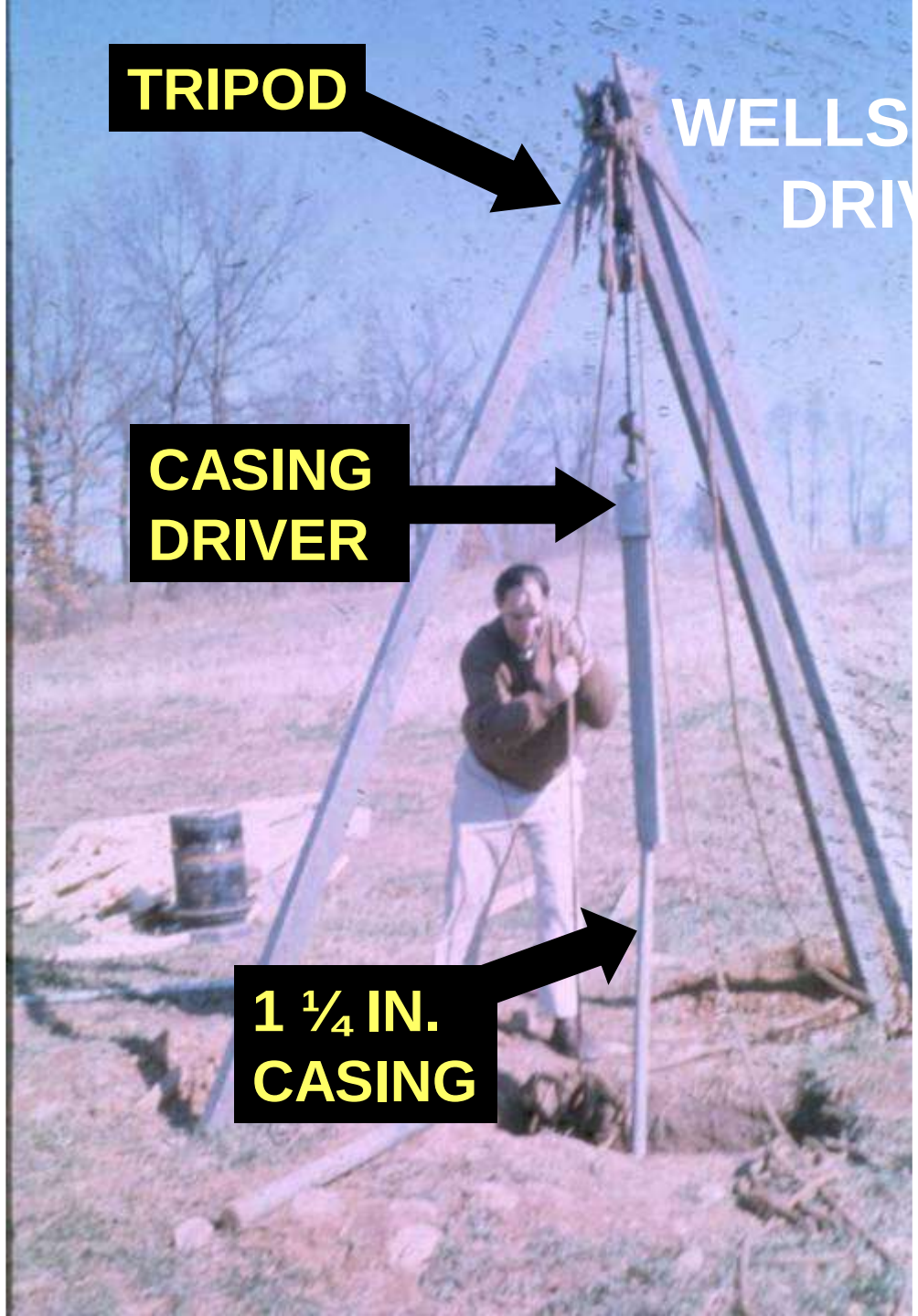
DRIVEN WELLS

- Installed in glacial drift only - CANNOT be driven through boulders or into bedrock
- Well point driven into ground with post-driver, tripod w/ weight or sledge hammer
- 1 1/4 in. to 2 in. diameter

DRIVEN WELLS

- Installed by property owners
- Common around lakes and high water table areas
- Most <35 ft. deep, limited yield (7 gpm or less)

MORE SUSCEPTIBLE TO SURFACE CONTAMINATION THAN DRILLED WELLS



WELLS BEING DRIVEN

DUG WELLS

- Large diameter (18-48 in.)
- Found in low yield areas
- Casing material - concrete crocks w/ loose joints

  Older wells: stones, brick-lined
- Water enters well through loose casing joints

DUG WELLS

- Older wells - hand dug

- Low well yield - storage in casing (100’s of gallons)

- **HIGHLY VULNERABLE TO CONTAMINATION**

SHALLOW UNSANITARY DUG WELL

OLD HAND-DUG WELL
LINED WITH FIELD STONE

Math-Sheet Civil '14

মো: রবিউল ইসলাম
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RUET

Water loss

1) The cumulative depth of infiltration in an experiment on a tube infiltrometer is observed to follow the equation $F = 0.165\, t^{0.65}$. Where F is in cm and t in minutes. Determine the equation for infiltration rate and the average infiltration rate.'13 '10 '09 (Reddi – 8.6; 269)

2) The rates of rainfall for the successive 30 min period of a 3 hr storm are 1.6, 3.6, 5.0, 2.8, 2.2, 1.0 cm/hr. The corresponding surface run-off is estimated to be 3.6 cm. Estimate the ϕ-index. Also determine the W-index.'06 '05 (Raghu – 3.6)

3) During a daily routine observation, 10.8 liters of water was added to bring the water surface in the evaporation pan to the stipulated level and the nearby raingauge measured 3.6mm of rainfall. What was the evaporation recorded for the day if the diameter of the pan is 122 cm? '10 (Reddi – 7.4; 231)

4) For a small catchment, the infiltration rate at the beginning of rain was observed to be 90 mm/hr. and decreased exponentially to a constant rate of 8 mm/hr. after 2.5 hr. The total infiltration during 2.5 hr. was 50 mm. Develop the Horton's equation for the infiltration rate at any time t<2.5 hr. '11 '15(example 3.5, roghu)

5) The infiltration capacity in a basin is represented by Horton's equation as $f_p = 3 + e^{2t}$, where f_p is in cm/h and t in hours. Assuming the infiltration to take place at capacity rates in a storm of 60 minutes duration. Estimate the depth of infiltration in (i) first 30 minutes and (ii) the second 30 minutes of the storm.'12

6) The infiltration capacity in a basin is represented by Horton's equation as $f_p = 6 + 16e^{2t}$, where f is in mm/h and t in hours. What are the values of initial filtration rate f_0, final constant rate f_c and constant K? if a storm occurs in this basin with an intensity of 22 mm/hr, determine the depth of infiltration for the 45 minutes and the average infiltration for the second 75 minutes.'14 '15 (Reddi – 8.7)

7) Calculate the amount of water lost by evaporation if: (i) Amount of precipitation = 12 mm (ii) quantity of surface inflow = 120 mm (iii) ground water flow amount = 75 mm (iv) surface outflow = 13 mm and (v) storage decreases by 5 mm.'14

8) Hourly rainfall of 2, 5, 6 and 3 cm over a 20 ha area consisting 4 ha of $\varphi = 5\ cm/hr$, 10 ha of $\emptyset = 3\ cm/hr$ and 6 ha of $\emptyset = 1\ cm/hr$. Derive hourly values of net rain. '14

ragibce 13

Runoff

10 9) A basin has an area of 26560 km^2, perimeter 965 km and length of thalweg 230 km. Determine (i) Form factor (ii) Elongation ratio (iii) Circularity ratio. '11 '10 '07

10) A drainage basin has an area of 210 Km^2. The average depth of rainfall received by it during a monsoon period is estimated to be $5.68 \times 10^7\ m^3$. Compute the depth of runoff. What percentage of rainfall has become runoff? If all this runoff volume is stored and used to irrigate a crop which requires 60 cm of water, how many hectors can be irrigated?'08

Average 5

11) The total observed runoff volume during a storm of 6 hr. duration with a uniform intensity of 15 mm/hr. is 21.6 $M.m^3$. If the area of the basin is 300 km^2, find the average infiltration rate and the runoff coefficient for the basin.'07

'15 12) The mean daily flows at a gauging station for a period of 7 days are 7, 27, 58, 41, 31, 20 and 13m^3/sec ,respectively. What is the total volume of stream flow in hectare-metres? If the drainage area at the site is 100 km^2, what is the runoff volume in cm?'15

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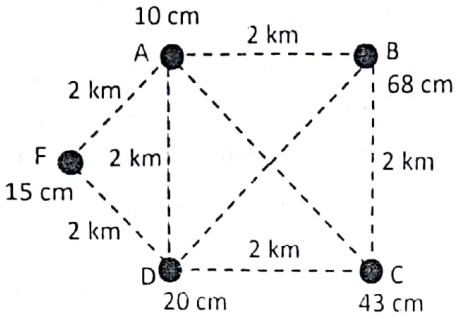
Precipitation

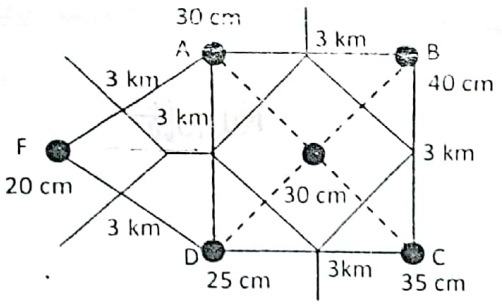
13) Find the mean precipitation for the area shown in figure below by Thiessen polygon method. The area is compound of a square plus an equilateral triangle plot of side 2km. Rainfall readings are in cm at the various



station indicated.' 10

14) Station X failed to report the rainfall recorded during a storm. With respect to east west and north south axes set up at station X, the coordinates of 4 surrounding gauges, which are the nearest to station X in the respective quadrants are (10,15), (-8,5), (-12,-9) and (5,-15) km respectively. Determine the missing rainfall at X, if the storm rainfalls at the four surrounding gauges are 73, 89, 68 and 57mm respectively. '09

15) Find the mean precipitation for the area shown in figure below by Thiessen polygon method. The area is compound of a square plus an equilateral triangle plot of side 3km. Rainfall readings are in cm at the various



ragibce 13

station indicated.'09 '06

16) A watershed has a network of rain gauges. Annual rainfall record by these gauges is given for a year as

Rain gauge	1	2	3	4	5
Annual rainfall	50	82	73	64	105

Calculate optimum number of rain-gauges for this water shed for a 10% error in estimate of mean annual rainfall.' 11 '08

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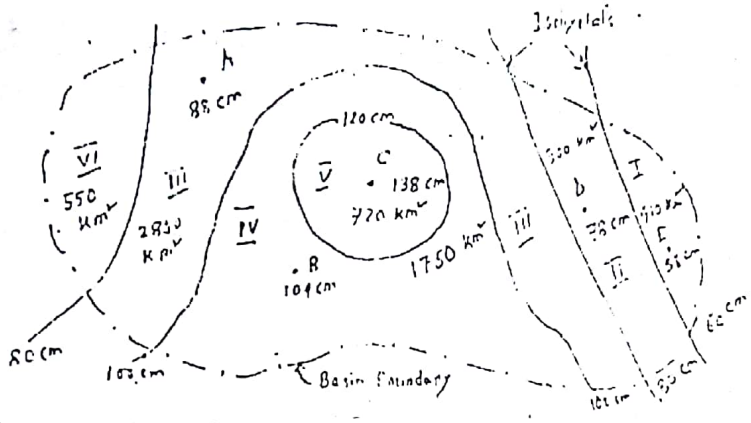
17) For the basin shown in figure below, the normal annual rainfall depths recorded and the isohytals are given. Determine the optimum number of rain-gauge stations to be established in the basin if it is desired to limit the error in the mean value of rainfall to 10%. Indicate how you are going to distribute the additional raingauge stations required, if any. What is the percentage accuracy of the existing network in the



estimation of the average depth of rainfall over the basin? '07

18) Consider a rectangular area whose (x,y) co-ordinates are (0,0), (4,0), (0,4) and (4,4). The area is 4 km wide and 4 km long (one-unit co-ordinate represents 1km). This area has 4 rain gauges. The locations of the gauges and the rainfall amounts measured by them are as follows: '05

Rainfall	Gauge co-ordinates (x,y)(km,km)	Rainfall amount (cm)
A	(1,1)	5
B	(1,3)	10
C	(3,3)	8
D	(3,1)	12

Determine the mean rainfall for this area by the Thiessen Polygon method.

ragibce 13

19) The annual rainfall at a place for a period of 19 years from 1970 to 1988 in mm are given as 520,615,420,270,305,380,705,600,350,550,560,400,520,435,395,290,430,1020 and 900. Construct the frequency curve and hence find the 75% and 50% dependable rainfall. What is the probability that a rainfall 800mm or more occurs in any year? '08 '06

20) A major river basin is divided into four sub basin with areas 920, 705, 1075, and 1665 km^2. If the average annual rainfall of these sub basins is 73, 85, 112, 100cm. respectively. What is the annual average rainfall for the basin as a whole? '11

21) A catchment has six raingauge stations. In a year, the annual rainfall is recorded by gauges as follows: '13

Station	A	B	C	D	E	F
Rainfall (cm)	82.6	102.9	180.3	110.3	98.8	136.7

For a 10% error in the estimation of mean rainfall. Calculate the optimum number of stations of catchment.

22) The mean daily flows at a gauging station for a period of 7 days are 7, 27, 58, 41, 31, 20 and 13 m^3/sec ,respectively. What is the total volume of stream flow in hectare-metres? If the drainage area at the site is 100 km^2, what is the runoff volume in cm? '15

23) Neighboring rain gauge stations, A, B, C, D, E and F have normal rainfalls of 610, 554, 468, 606, 563 and 382 mm respectively. During a storm stations A, B, C, D, E and F have reported rainfalls of 22, 29, 35, 13 and 25 mm respectively, and station D did not report as it was inoperative. Estimate the missing storm rainfall at D by the arithmetic average and normal ratio method. '14

58

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24) A rainfall of certain high intensity is expected to occur once in 20 years. What is the chance of occurrence in any year? What is the probability that it may occur in the next 12 years? '15

Stream Gauging

25) Following vehicles were recorded in a station with a current meter. '10 '06

Depth above bed(m)	0	1	2	3	4
Velocity (m/sec)	0	0.5	0.7	0.8	0.8

Find the discharge per unit width of stream near the point of measurement. Depth of flow at the point was 5 meter.

26) Following velocities were recorded in a stream with current meter.'12

Depth above bed(m)	0	0.7	1.4	2.1	2.8
Velocity (m/sec)	0	2.62	2.94	3.16	3.28

Find the discharge per unit width of stream near the point of measurement by two-point method. Depth of flow at the point was 3.5 meter.

27) Following vehicles were recorded in a station with a current meter. velocities

ragibce 13

Depth above bed(m)	0	1	2	3	4
Velocity (m/sec)	0	0.8	1.0	1.2	1.2

Find the discharge per unit width of stream near the point of measurement. Depth of flow at the point was 5.0 m.'13

28) Compute the value of stream flow rate, with the help of following data using the two-point method.'08

Depth from one end(m)		0	1	2	3	4	5	6	7
Depth(m)		0	1.4	3.3	5.0	9.0	5.4	3.8	1.8
Velocity at (m/s)	0.2d	0	0.40	0.60	0.84	0.90	0.80	0.62	0.54
	0.6d	0	0.25	0.35	0.60	0.70	0.65	0.50	0.36
	0.8d	0	0.20	0.30	0.50	0.62	0.55	0.40	0.30

Hydrograph

29) The followings are the ordinates of a 3-hour unit hydrograph. Derive the ordinates of 6-hour unit hydrograph and plot the same.'10

Time(hr.)	0	3	6	9	12	15	18	21	24
3-hr UGO (cumec)	0	1.5	4.5	8.6	12.0	9.4	4.6	2.3	0.8

30) The unit ordinates of 3-hr hydrograph are given below. Find out the ordinates of 9-hr unit hydrograph. '12 '14

Time (hr.)	0	3	6	9	12	15	18	21
3-hr. OUH (m^3/s)	0	50	75	150	80	50	30	0

31) The ordinate of four-hour unit hydrograph is given below. Derive the ordinates of 8-hr. unit hydrograph by the S-curve method. '11 '06

Time (hr.)	0	4	8	12	16	20	24	28	32	36	40	44
4-hr. unit hydrograph (cumec)	0	24	82	159	184	151	103	64	36	17	6	0

	1-3-1999								2-3-1999							
Time(min)	2	5	8	11	14	17	20	23	2	5	8	11	14	17	20	23

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BFO = Base flow ordinate TRO = Total Runoff ordinate
DRO = Direct " "

Discharge(cumec)	50	47	75	120	225	290	270	145	110	90	80	70	60	55	51	51

32) The runoff data as a stream gauging station for a flood are given below. The drainage area is 40 km^2. The duration of rainfall is 3 hrs. Derive the (three) 3-hr unit hydrograph for the basin.'08

33) The run off data at a stream gauging station for flood are given below. The drainage area is 40 km^2 the duration of rainfall is 3 hrs. Derive the three-hr. unit hydrograph for the basin and plot the same. '11

	1-3-1970								2-3-1970							
Date & Time (hr.)	2	5	8	11	14	17	20	23	2	5	8	11	14	17	20	23
Discharge (cumec)	50	47	75	120	225	290	270	145	110	90	80	70	60	55	51	50

34) A steady 6h rainfall with intensity of 4 cm/hr. produces a peak discharge of 560 cumec. The average storm loss can be assumed as 1cm/hr. and base flow 20 cumec. What is the peak discharge of unit hydrograph and its duration? On the same basin determine the discharge from a 6h rainfall at intensity of 3.5 cm /hr., assuming an average loss rate of 1.5cm/hr. and base flow of 1.5 cumec.'08

35) The design storm over a water shed has depth of rainfall of 4.6, 3.6 and 5.6 cm in successive 1 hr. periods. The 1 hr. UG can be approximated by a triangle of base 9 hr. with a peak cumec occurring 2 hr. from the beginning. Compute the flood hydrograph assuming an average loss rate of 6 mm/hr. and constant base flow of 5 cumec. What is the area of water shed and its coefficient of runoff? '07

36) Compute the peak of a 4 hr. UG if the following data are available. The effective rainfall of 4 h duration has produced a flood hydrograph peak of 400 m^3/sec. This flood hydrograph has a base flow of 40 m^3/s. The average watershed rainfall is 4 cm, and average loss rate is 0.4 cm/hr.'05

37) The design storm of a watershed has a depth of rainfall 4.9 and 3.9 cm for the consecutive 1 hr. period. The 1 hr UG can be approximated by a triangle of base 6 hr. with a peak of 50 cumec occurring after 2 hr. from the beginning. Compute the flood hydrograph assuming an average loss rate of 9 mm hr and constant base flow of 10 cumec.'13 '15 What is the area of watershed and its coefficient of runoff?'05

38) The peak of flood hydrograph due to a 3 hr. duration isolated storm in a catchment is 270 m^3/s. The total depth of rainfall is 5.9 cm. Assuming an average infiltration loss of 0.3 cm/hr. and a constant base flow of 20 m^3/s. Estimate-

(i) The peak of the 3-h unit hydrograph (UH) of this catchment

(ii) If the area of the catchment is 567 km^2, determine the base width of the 3-h unit hydrograph by assuming it to be triangular in shape.'05

Ground Water

R-5.1 39) A 30cm well fully penetrates into a confined aquifer 30m deep. After a long period of pumping at a rate of 1200 lpm, the drawdown in the wells 20m and 45m from the pumping well are found to be 2.2m and 1.8m respectively. Determine the transmissibility of the aquifer. What is the drawdown in the pumped well?' 10 '08 '07

R-5.2 40) A 30cm well fully penetrates a confined aquifer 50m deep. After a long period of pumping at a rate of 1800 lpm, the drawdown in the wells at 15 and 45m from the pumping well are found to be 1.7 and 0.8 respectively. Determine the transmissibility of the aquifer. What is the drawdown in the pumped well?'06

41) A 25 cm well penetrates 30m below static water level (GWT). After a long period of pumping at rate of 1800 lmp. The draw down in observation well at 13 m and 38 m from the pumped well are 1.2 m and 0.5m respectively. Determine i) The transmissivity of the aquifer ii) the draw down in pumped well assuming $R = 300\ m$.'11

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42) In a homogeneous isotropic confined aquifer of constant thickness of 30 m, specific retention of 4%. Hydraulic conductivity of 20 m/day and specific yield of 16%, two observation wells 1000 m apart indicates piezometric head of 7.6m & 5.4m respectively above M.S.L. Determine the actual velocity in pores and Reynolds number for the flow assuming uniform flow, average grain diameter of sand 1 mm and $\nu_{water} = 0.01\ cm^2/s$.'12

43) A well with a radius of 0.5m, completely penetrates an un-confined aquifer of thickness 50m and K=30m/day. The well is pumped so that the water level on the well remains at 40m above the bottom. Assuming that pumping has essentially no effect on water table at r=500m. What is the steady rate discharge?'09

44) A 20cm diameter well penetrates fully a confined aquifer of thickness 25m. When the well pumped at a rate of 200 lit/min, the steady state drawdown in the two observations wells located at 10m and 100m distance from the pumping well are found to be 3.5m and 0.05m respectively. Calculate the permeability and transmissibility of the aquifer.'08

45) In a certain place the average thickness of the confined aquifer is 30m and extends over an area of 800 km^2. The piezometric surface fluctuates annually from 19m to 9m above the top of the aquifer. Assume a storage coefficient of 0.0008. What ground storage can be expected annually?'06

46) Assuming an average well yield of 30 m^3/hr and about 200 days of pumping in a year, how many wells can be drilled in the area?'06

47) In an area of 120 ha, the water table dropped by 5m. If the porosity is 28.0% and the specific retention is 9.0%. Determine the specific yield of the aquifer and change in ground water storage.'09

48) An artesian aquifer 30 m thick has a porosity 25% and bulk modulus of compression 2000 kg/cm^2. Estimate the storage coefficient of the aquifer. What fraction of this is attributable to expansibility of water? Assume, Bulk modulus of elasticity of water $2.4 \times 10^4\ kg/cm^2$.'11 '14

49) A 30 cm well penetrating a confined aquifer is pumped at a constant rate of 700 lpm. The draw down at an observation well at a radial distance of 100 m is as follows'12

Time in days	0.001	0.005	0.01	0.05	0.10	0.50	1.0	5.0	10.0
Drawdown in m	0.083	0.196	0.239	0.376	0.431	0.559	0.614	0.742	0.797

Determine the aquifer parameters using the Chow's method.

50) A well of 0.5 m diameter penetrates fully into a confine aquifer is pumped at a constant rate of 1200 lpm. The drawdown at an observation well at a radial distance of 30 m is as follows:'13

Time (min)	1.0	2.5	5	10	20	50	100	200	500	1000
Drawdown (m)	0.2	0.5	0.8	1.2	1.8	2.5	3.0	3.7	4.4	5.0

Calculate the aquifer parameters by using Cooper and Jacob method.

51) In an area of 100 ha the water table dropped by 4.5 m. if the porosity is 30 % and the specific retention is 10%, determine (i) the specific yield of the aquifer and (ii) change in ground water storage. '15

52) A well is pumped at the constant rate of 8 litres/sec. A match of the well function [u versus w(u)] with drawdown versus time data from an observation well located at a distance of 450 m from the pumped well has produced the following matching values: u=1, w(u) = 1, s = 0.21 m, $r^2/t = 2055\ m^2/min$. Calculate the transmissibility and storage coefficient of aquifer. Also estimate the drawdown in the well at the end of 10 days of pumping.'14 '15

u	W(u)
6×10^{-3}	4.54
7×10	4.39

53) A well of 0.5 m diameter penetrates fully into a confined aquifer of thickness 20 m and hydraulic conductivity $8.2 \times 10^{-1}\ m/s$. What is the maximum yield from this well if the draw down in well is not to exceed 3 m? The radius of influence may be taken as 200 m. '13

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Flood Routing

54) During the construction period of 10 years of a reservoir, a cofferdam is required to be constructed with a capacity to take care if 5 years floods. What is the probability that (i) the flood will not occur at all and (ii) it will occur twice during the construction period? '12

55) From the analysis of available data of annual flood peaks of a small stream for a period of 35 years, the 50 year and 100-year flood have been estimated to be 660 m^3/sec and 740m^3/sec using Gumbel's method. Estimate the 200-year flood for the stream. The values of mean y_n and standard deviation σ_n for different period of flood data are as follows: '10

n	35	50	100
$\overline{y_n}$	0.54034	0.54854	0.56002
σ_n	1.12847	1.16066	1.20649

56) A river is dry for 5% days in a year. A crop requires 115 days water supply from this river for its base period of 120 days. What is the probability that the crop will fail in a year? What is the probability of failure in 4 years? '09

57) The analysis of annual flood peaks of a stream with 60 years of record indicates that the mean value of the annual peaks is 2264 m^3/sec. The standard of the peaks is 340 m^3/sec. '09

(i)Find the probability of having a flood next year of magnitude equal to or greater than 3170 m^3/sec

(ii)What is the probability that at least one flood of such magnitude will occur during the next 25 years.

58) The annual peak flood discharges recorded at a stream gauge site for 19 years during the period of 1961 to 1979 in m^3/sec is given below.
3950,6790,7660,4220,2820,5600,7050,5280,5200,4360,6970,6240,4960,5890,5980,3590,6860,7210,5270.
Construct the frequency curve and hence find the flood peak with a return period of 50 years and 100 years. '09

59) The analysis of a 30-year flood data at a point in a river yielded x=1200 m^3/sec and S_x=650 m^3/sec. For what discharge would you design the structure at this point to provide 95% assurance that the structure would not fail in the next 50 years for n=30, y_n=0.53622 and ∂_n=1.11238 '07

60) The analysis of 30 year flood data at a point on a river yielded $\bar{x} = 1200\ m^3/s$ and $S_x = 650 m^3/s$. For what discharge would you design the structure at this point to provide 95% assurance that the structure would not fail in the next 50 years? The values of $\overline{y_n}$ and σ_n for a record length of n years are given below: '13 '15 '07

n	30	35	50	100
$\overline{y_n}$	0.53622	0.54034	0.54854	0.56002
σ_n	1.11238	1.12847	1.16066	1.20649

61) From the analysis of available data on annual flood peaks of a small stream for a period of 35 years, the 50 year and 100 year flood have been estimated to be $660 m^3/s$ and $740\ m^3/s$ using Gumbel's method. Estimate the 150-year flood for the stream. The value of $\overline{y_n}$ and σ_n for a record length of n years are given below '12

Time (h)	0	12	24	36	48	60	72	84	96	108	120	132	144	156	168	180
Inflow(m^3/s)	22	35	78	262	332	278	230	188	152	123	100	80	69	58	51	46

62) Route the flood hydrograph given below through a channel reach and derive the outflow hydrograph. The value of k and x for the reach may be taken as 12h and 0.278 respectively. '07

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Time (h)	0	4	8	12	16	20	24	28	32	36	40	44	48	52	56
Inflow(m^3/s)	42	68	116	164	194	200	192	170	150	128	106	88	74	62	54

63) If the atmospheric pressure, temperature and vapor pressure of the atmosphere are 1005 mb, 28° C and 19.873 mm of mercury. Determine relative humidity, dew point temperature and the density of the atmosphere. Given for 28^0 C, e_s=37.796 mb.

64) A surface float took 10 sec to travel a straight run of a stream of 20 m. What is appropriate mean velocity of the stream? If a velocity rod had been used, what time it would have taken to travel the same run?

ragibce 13

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Precipitation

Raghunath:

Estimating the missing storm Rainfall

Example-2.1:

Here, $P_A = 8.5$ cm, $N_A = 75$ cm

$P_B = 6.7$ cm, $N_B = 84$ cm

$P_C = 9.0$ cm, $N_C = 70$ cm

$P_D = ?$, $N_D = 90$ cm

(i) $\frac{N_D - N_A}{N_D} \times 100 = \frac{90-75}{90} \times 100 = 16.67\% > 10\%$

(ii) $\frac{90-84}{90} \times 100 = 6.67 < 10\%$

(iii) $\frac{90-70}{90} \times 100 = 22.2 > 10\%$

∴ Average value of P_D have to calculated in Normal ratio method.

Now, $P_D = \frac{1}{m}\left[\frac{N_D}{N_A} \times P_A + \frac{N_D}{N_B} \times P_B + \frac{N_D}{N_C} \times P_C\right]$

$= \frac{1}{3}\left[\frac{90}{75} \times 8.5 + \frac{90}{84} \times 6.7 + \frac{90}{70} \times 9\right]$

$= 9.65$ cm

(Ans:)

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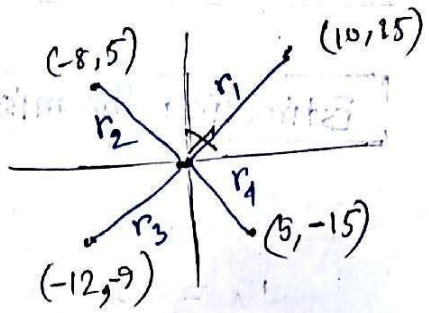
Reddi

Sheet — 19

(2009)

Example-5.13: (Page-98)



Given,

$P_1 = 73$ mm

$P_2 = 89$ mm

$P_3 = 68$ mm

$P_4 = 57$ mm

$r_1^2 = 10^2 + 15^2 = 325$

$r_2^2 = (-8)^2 + 5^2 = 89$

$r_3^2 = (-12)^2 + (-9)^2 = 225$

$r_4^2 = (5)^2 + (-15)^2 = 250$

National Weather service method,

We know,

$$P_x = \frac{\frac{P_1}{r_1^2} + \frac{P_2}{r_2^2} + \frac{P_3}{r_3^2} + \frac{P_4}{r_4^2}}{\frac{1}{r_1^2} + \frac{1}{r_2^2} + \frac{1}{r_3^2} + \frac{1}{r_4^2}}$$

$$= \frac{\frac{73}{325} + \frac{89}{89} + \frac{68}{225} + \frac{57}{250}}{\frac{1}{325} + \frac{1}{89} + \frac{1}{225} + \frac{1}{250}}$$

$= 77.11$ mm (Ans)

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Sheet – 23 (2014)

Given,

$N_A = 610$ mm $P_A = 22$ mm

$N_B = 559$ mm $P_B = 29$ mm

$N_C = 468$ mm $P_C = 35$ mm

$N_D = 606$ mm $P_D = ?$

$N_E = 563$ mm $P_E = 13$ mm

$N_F = 382$ mm $P_F = 25$ mm

$\frac{N_D - N_A}{N_A} = \frac{-606 + 610}{} = 0.67 < 10\%$

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Example-2.4 (Page-32)

Optimum Raingauge determination

(2013, 2008, 2007, 2011)

* Sheet-16, 21

Station	Normal Rain fall, x	$\bar{x} = \frac{\Sigma x}{n}$	Difference $(x - \bar{x})$	(Difference)^2 $(x - \bar{x})^2$
A	88		$(88 - 92.8) = -4.8$	23
B	104		11.2	125.4
C	138	92.8	45.2	2040
D	78		-14.8	219
E	56		-36.8	1360
n=5	$\Sigma x = 464$			$\Sigma(x_i - \bar{x})^2 = 3767.4$

$$\therefore \sigma_n = \sqrt{\frac{\Sigma(x-\bar{x})^2}{n-1}} = \sqrt{\frac{3767.4}{5-1}} = 30.7$$

$$C_v = \frac{\sigma}{\bar{x}} \times 100 = \frac{30.7}{92.8} \times 100 = 33.1\%$$

Given, $P = 10\%$. $\therefore$ Number of raingauge, $N = \left(\frac{C_v}{P}\right)^2$

$$= \left(\frac{33.1}{10}\right)^2 = 11$$

Additional raingauge sations to be established $= (N - n) = (11 - 5)$

$= 6$

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Additional 6 raingauge have to be distributed :

Zone	I	II	III	IV	V	VI	Total
Area (km^2)	410	900	2850	1750	720	550	7180
$\% A = \frac{Ai}{\Sigma A}$	0.06	0.12	0.40	0.24	0.10	0.08	1.00
N × % Area	0.66	1.32	4.4	2.64	1.1	0.88	
Rounded Value	1	1	4	3	1	1	11
Raingage	1	1	1	1	1	—	5
Additional rain gauge	—	—	3	2	—	1	6

$$\text{Percentage error} = \frac{C_v}{\sqrt{N}} = \frac{33.1}{\sqrt{5}} = 14.8\%$$

$$\text{Percentage Accuracy} = (100 - 14.8) = 85.2\%$$

Ans

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Sheet-16, 21
(2011, 08)

Raingauge	Annual Rainfall (x_i)	$\bar{x} = \frac{\Sigma x}{n}$	$x_i - \bar{x}$	$(x_i - \bar{x})$
1	50		-24.8	615.04
2	82		7.2	51.84
3	73	74.8	1.8	3.24
4	69		-10.8	116.64
5	105		30.2	912.04
$n=5$	$\Sigma x = 374$			$\Sigma(x_i - \bar{x}) = 1698.8$

$$\therefore \sigma = \sqrt{\frac{\Sigma(x_i - \bar{x})^2}{n-1}} = \sqrt{\frac{1698.8}{5-1}} = 20.61$$

$$\therefore C_v = \frac{\sigma}{\bar{x}} = \frac{20.61}{74.8} = 0.28$$

We know, No. of raingage, $N = \left(\frac{C_v}{E_p}\right)^2$ Here, $E_p = 10\%$

$$N = \left(\frac{0.28}{0.1}\right)^2 = 7.84 \simeq 8$$

$\therefore N = 8$

(Ans.)

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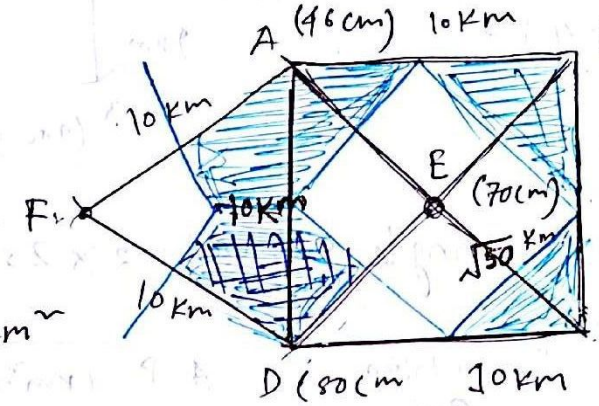
Determination of Mean Areal Precipitation

Example-2.3(a) - (Raghunath) - (Page-29)

Sheet-15

(2009, 06)



(polygon Method)

Area of sq. plot
$= 10^2 = 100\ km^2$

∴ inner sq. plot
area $= (\sqrt{50})^2 = 50\ km^2$

Area of each corner Triangle in sq. root

$$= \frac{(100-50)}{4} = \frac{50}{4}\ km^2 = 12.5\ km^2$$

$\frac{1}{3}$ area of equilateral triangle plot $= \frac{1}{3} \times (2 \times 10 \times 10 \sin 60^\circ)$

$= 14.4\ km^2$

Station	Area, A (km^2)	Precipitation P (cm)	A × P (km^2-cm)
A	(12.5+14.4) = 26.9	46	1238
B	12.5	65	813
C	12.5	76	950
D	26.9	80	2152
E	50	70	3500
F	14.4	60	864
n=6	ΣA=143.2	ΣP=397	ΣA·P=9517

P_{ave} & Arithmatic mean

$$P_{ave} = \frac{\Sigma A \cdot P}{\Sigma A} = \frac{9517}{143.2} = 66.3\ cm$$

Arithmatic mean

$$= \frac{\Sigma P}{n} = \frac{397}{6} = 66.17\ cm.$$

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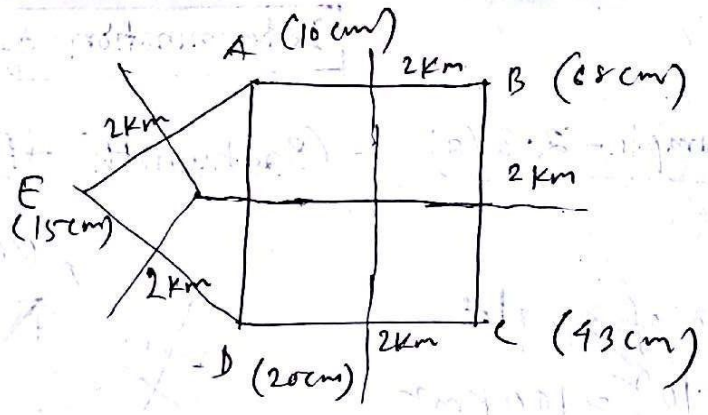
Sheet-13 (polygon method)

(2004, 2010)



Area of sq. plot = $2^2 = 4$ km^2

$\therefore \frac{1}{4}$ of 11 = $\frac{4}{4} = 1$ km^2

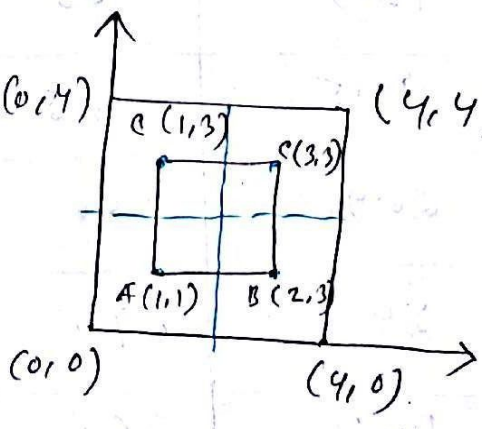
$\frac{1}{3}$ Area of equilateral triangle = $\frac{1}{3} \times \frac{1}{2} \times 2 \times 2 \sin 60° = 0.58$ km^2

Station	Area, A (km^2)	Precipitation (cm)	A P (km^2-cm)	
A	1+0.58 = 1.58	10	15.8	$P_{ave} = \frac{\Sigma AP}{\Sigma A}$
B	1	68	68	$= \frac{167.1}{5.74}$
C	1	43	43	= 29.11 cm
D	1.58	20	31.6	Arithmatic mean,
E	0.58	15	8.7	$= \frac{\Sigma P}{n} = \frac{156}{5}$
	$\Sigma A = 5.74$	$\Sigma P = 156$	$\Sigma A.P = 167.1$	= 31.2 (m

Sheet-18

(2005) Area of total sq. plot = $4^2 = 16$ km^2

Area of each sq. plot = $\frac{16}{4} = 4$ km^2



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Station	Area, A (km^2)	P (cm)	AP (km^2-cm)
A	4	5	20
B	4	10	40
C	4	8	32
D	4	12	48
	$\Sigma A = 16$		$\Sigma AP = 140$

$$P_{av} = \frac{\Sigma AP}{\Sigma A}$$

$$= \frac{140}{16}$$

$$= 8.75 \text{ cm}$$

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Water Losses

Example-3.1 (Raghunath) Page-63
(2007)

Mean water spread area of lake → (From cone formula)

$$A_{ave} = \frac{1}{3}(A_1 + A_2 + \sqrt{A_1 A_2})$$

$$= \frac{1}{3} \times (2.80 + 2.55 + \sqrt{2.8 \times 2.55})$$

$$= 2.673 \text{ km}^2$$

Annual loss of water due to evaporation,

$$= (16.7 + 14.3 + 17.8 + 25.0 + 28.6 + 21.4 + 16.7 + 16.7 + 16.7 + 21.4 + 16.7 + 16.7)$$

$$= 228.7 \text{ cm.}$$

∴ Annual volume of water lost due to evaporation,

$$= (2.673 \times 10^6) \times \frac{228.7}{100} \times 0.7$$

$$= 4.129 \times 10^6 \text{ m}^3$$

$$= 4.29 \text{ M m}^3$$

(Ans.)

Example-3.1 (6) (Raghunath) – Page-64

Rain fall, $p = (15 \times 6) = 90$ mm

Runoff, $R = \frac{21.6 \times 10^6}{300 \times 10^6} = 0.072 \text{ m} = 72 \text{ mm}$

∴ Infiltration loss $F_p = (P - R) = (90 - 72) = 18$ mm

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$\therefore$ Infiltration rate, $f_{ave} = \frac{F_p}{t} = \frac{18}{6} = 3$ mm/hr

We know,

Yield = C × A × P

$\Rightarrow 21.6 \times 10^6 = c \times 300 \times 10^6 \times \frac{90}{1000}$

$\therefore c = 0.8$

(Ans.)

Example-3.4 (2) (Raghunath) – Page-77 (11,15 **Sheet-4**

Given, At beginning, $f_0 = 90$ mm/hr, $t = 2.5$ hr

At final, $f_c = 8$ mm/hr, Total infiltration, $F = 50$ mm.

We know, $f = f_c + (f_0 - f_c) \times e^{-Kt}$

Here, $K = \frac{f_0 - f_c}{F_c} = \frac{90 - 8}{50 - 8 \times 2.5} = 2.73 \text{ hr}^{-1}$

$\therefore$ Horton equation is,

$\therefore f = 8 + (90 - 8) \times e^{-2.73 \times t}$ where,

f in mm/hr

and t, in hr

(Ans.)

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Example – 3.5 (Raghunath – Page 77)

Given,

initial filtration rate, $f_0 = 1$ cm/hr

final filtration rate, $f_c = 0.3$ cm/hr

Total rain fall observed, $P = 10$ cm

$K = 5\ hr^{-1}$, $t = 24$ hr.

Now, Total infiltration, $F = \int_0^t [f_c + (f_0 + f_c)e^{-Kt}]\,dt$

$$= \int_0^{24} [(0.3 + (1-0.3)e^{-5t}]\,dt$$

$$= \left[0.3t + \frac{0.7}{(-5)e^{5t}}\right]_0^{24}$$

$$= \left(0.3 \times 24 - \frac{0.7}{5 \times e^{5\times 24}}\right) - \left(0.3\times 0 - \frac{0.7}{5e^0}\right)$$

$$= 7.34 \text{ cm}$$

Now,

Run off $= P - F - E$

$= 10 - 7.34 - (0.6 \times 0.7)$

$= 2.24$ cm.

∴ volume of runoff from the catchment.

$$= 1.8 \times 10^6 \times \frac{2.24}{100}$$

$$= 40320\ m^3$$ (Ans.)

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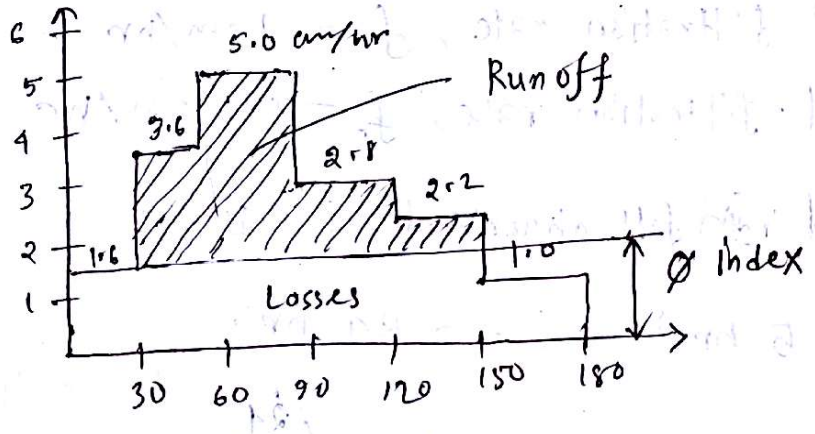
Example -3.6 (Raghunath) - Page-83 Sheet-2

(05, 06



Now,

Total rain fall, $P = (1.6 + 3.6 + 5.0 + 2.8 + 2.2 + 1.0)$ cm

$= 8.1$ cm.

Let, ϕ index will be with in (1 to 1.6 cm)

Given that, $P_{net} = 3.6$

Hence, $\Sigma(i - \phi)t = P_{net}$

$$\left[(1.6-\phi) + (3.6-\phi) + (5-\phi) + (2.8-\phi) + (2.2-\phi)\right] \times \frac{30}{60} = 3.6$$

$$\Rightarrow (15.2 - 5\phi) \times \frac{1}{2} = 3.6$$

$\Rightarrow \phi = 1.6$ cm/hr which is with in 1 to 1.6 cm/hr

Now, ϕ index $= \dfrac{P - Q - S}{t_R} = \dfrac{8.1 - 1.6 - 0}{3} = 1.5$ cm/hr

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Example - 3.7 (Raghunath) Page - 84 | Sheet - 8

(14, We know, $P_{net-mean} = \frac{\sum A_i P_{net-i}}{\sum A_i}$

1st hour, $P = 2.5$ cm.

$P_{net-mean} = \frac{4\times(0) + 10\times(0) + 6\times(2.5-1)}{20} = 0.45$ cm

2nd hour, $P = 6$ cm.

$P_{net-mean} = \frac{4\times(6-5) + 10\times(6-3) + 6\times(6-1)}{20} = 3.20$ cm

3rd hour, $P = 3$ cm

$P_{net-mean} = \frac{4\times(0) + 10\times(0) + 6\times(3-1)}{20} = 0.60$ cm

∴ Total net rain for 3 hour storm = (0.45 + 3.20 + 0.60)

= 4.25 cm. (Ans)

Reddi

Example - 8.2 (Page - 269) | Sheet - 1

(13, 10, 09

Given, $F = 0.165\, t^{0.65}$; F in cm and, t in minutes.

∴ infiltration rate, $f = \frac{dF}{dt} = \frac{d}{dt}(0.165\, t^{0.65})$

$= 0.165 \times 0.65\, t^{(0.65-1)}$

$= 0.10725\, t^{-0.35}$ cm/min

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$$\therefore f = (0.10725\, t^{-0.35} \times 60) \text{ cm/hr}$$

$$f = 6.435\, t^{-0.35} \text{ cm/hr}$$

Now,

$$f_{ave} = \frac{F}{t} = \frac{0.165\, t^{0.65}}{t} = 0.165\, t^{0.65-1} \text{ cm/min}$$

$$\therefore f_{ave} = 0.165\, t^{-0.35} \times 60 \text{ cm/hr}$$

$$f_{ave} = 9.9\, t^{-0.35} \text{ cm/hr}$$

(Ans)

Example- 8.7 (Reddi) Page-269 Sheet-6

(14, 15, 12)

Given, $f = 6 + 16 e^{-2t}$ mm/h ..... (I)

We know, Horton's equation, $f = f_c + (f_o - f_c) e^{-Kt}$ ..... (II)

comparing (I) & (II).

$f_c = 6$ mm/hr

$f_o - f_c = 16 \Rightarrow f_o = (16 + 6) = 22$ mm/hr

and, $K = 2 \text{ hr}^{-1}$

Now, For first 45 minutes, $F = \int_0^t f\, dt$

$$= \int_0^{\frac{45}{60}} (6 + 16 e^{-2t})\, dt$$

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$$\Rightarrow F = \left[6t + \frac{16}{-2e^{2t}}\right]_0^{0.75} = 10.715 \text{ mm}$$

Again,

For first 75 minutes, $F = \int_0^{\frac{75}{60}} (6 + 16e^{-2t})\,dt$

$$= 14.843 \text{ mm}.$$

$$f_{ave} = \frac{F}{t} = \frac{14.843}{1.25} = 11.874 \text{ mm/hr}$$

For second 75 minutes, $F = \int_{\frac{45}{60}}^{\frac{45+75}{60}} (6 + 16e^{-2t})\,dt$

$$= 9.14 \text{ mm}.$$

$$f_{ave} = \frac{F}{t} = \frac{9.14}{\left(\frac{75}{60}\right)} = 7.312 \text{ mm/hr}$$

(Ans.)

Example – 7.4 – (Reddi) Page - 230

Sheet – 3

(10

Given, Dia. of pan = 122 cm.

$\therefore$ Area of pan $= \frac{\pi}{4} \times (122)^2 = 11689.87 \text{ cm}^2$

Added volume of water = 10.8 Litres $= 10.8 \times 10^3 \text{ cm}^3$

$\therefore$ Depth of water added $= \frac{\text{volume}}{\text{Area}} = \frac{10.8 \times 10^3}{11689.87}$ cm

$= 0.924$ cm.

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Given

Rain fall = 3.6 mm = 0.36 cm

Now, we know, Evaporation = depth of water added + rainfall

= 0.924 + 0.36

= 1.284 cm. (Ans)

Sheet-7

2K(14)

Given, P = 12m O = 13 mm

I = 120m S = 5 mm

Og = 75 mm

we know,

$$P + I \pm Og = O + E \pm S$$

$$\Rightarrow 12 + 120 - 75 = 13 + E - 5$$

$$\Rightarrow E = 49 \text{ mm}$$

(Ans)

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Runoff

Example-4.1 (Raghunath)- Page-150

Given, N_c in Horizontal = 126 ; Total grid in Horizontal = 55250 m

N_c in vertical = 75 ; Total grid in vertical = 53260 m

C.I = 25.

Slope in Horizontal direction,

$$S_H = \frac{N_c \times C.I}{\Sigma H} = \frac{126 \times 25}{55250} = 0.0570 \text{ m/m}$$

Slope in vertical direction,

$$S_V = \frac{N_c \times C.I}{\Sigma V} = \frac{75 \times 25}{53260} = 0.0352 \text{ m/m}$$

∴ Mean slope of the basin,

$$S = \frac{S_H + S_V}{2} = \frac{0.0570 + 0.0352}{2}$$

$$\therefore S = 0.0461 \text{ m/m} = 4.61\%$$

Also, from Horton's equation,

$$S = \frac{1.5 \times (C.I) \times N_c}{\Sigma L} = \frac{1.5 \times 25 \times (75+126)}{(53260 + 55250)}$$

$$\therefore S = 0.0695 \text{ m/m} = 6.95\%$$

(Ans.)

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Example- 4.2 (Raghunath) – (Page- 101)

(07, 10, 11)

Given that, $A = 26560\ km^2$

$P_b = 965\ km$

$L_b = 230\ km.$

(i) Form factor, $F_f = \frac{W_b}{L_b} = \frac{A}{L_b^2} = \frac{26560}{(230)^2} = 0.502$

(ii) Compactness Co-efficient, $C_c = \frac{P_b}{2\pi R} = \frac{P_b}{2\sqrt{\pi A}} = \frac{965}{2 \times \sqrt{3.1416 \times 26560}}$

$\therefore C_c = 1.67$

(iii) Elongation Ratio, $E_r = \frac{2R}{L_b}$

Here, $\pi R^2 = A$

$\Rightarrow R = \sqrt{\frac{A}{\pi}} = \sqrt{\frac{26560}{3.1416}} = 91.95\ km$

$\therefore E_r = \frac{2 \times 91.95}{230} = 0.8$

(iv) Circularity ratio, $C_r = \frac{A}{\pi R'^2}$

$2\pi R' = P_b$

$\therefore R' = \frac{965}{2 \times 3.1416} = 153.584$

$\therefore C_r = \frac{26560}{3.1416 \times (153.584)^2}$

$\therefore C_r = 0.3584$

(Ans.)

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Example - 4.3 (Raghunath) — Page - 4.3

Subarea contributed runoff (ha)	Time beginning from the storm (hr)						
	1	2	3	4	5	6	7
I	20	20	20	20			
II		30	30	30	30		
III			50	50	50	50	
IV				40	40	40	40
Discharge, $Q = CAP$	$0.5 \times (20 \times 10^4) \times \frac{1}{100}$	$0.6 \times (20 \times 10^4) \times \frac{1}{100} + 0.5 \times (30 \times 10^4) \times \frac{1}{100}$	$0.7 \times (20 \times 10^4) \times \frac{1}{100} + 0.6 \times (30 \times 10^4) \times \frac{1}{100} + 0.5 \times (50 \times 10^4) \times \frac{1}{100}$	$0.8 \times (20 \times 10^4) \times \frac{1}{100} + 0.7 \times (30 \times 10^4) \times \frac{1}{100} + 0.6 \times (50 \times 10^4) \times \frac{1}{100} + 0.5 \times (40 \times 10^4) \times \frac{1}{100}$	$0.8 \times (30 \times 10^4) \times \frac{1}{100} + 0.7 \times (50 \times 10^4) \times \frac{1}{100} + 0.6 \times (40 \times 10^4) \times \frac{1}{100}$	$0.8 \times (50 \times 10^4) \times \frac{1}{100} + 0.7 \times (40 \times 10^4) \times \frac{1}{100}$	$0.8 \times (40 \times 10^4) \times \frac{1}{100}$
ΣQ	1000	2700	5700	8700	8200	6800	3200

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Sheet-10 | Reddi-6.8 Example

2K(08)

Given, Runoff Volume $= 5.68 \times 10^7\ m^3$

Area of basin $= 210\ km^2$

$\therefore$ Depth of runoff $= \dfrac{5.68 \times 10^7}{210 \times 10^6} = 0.27\ m$

$= 27\ cm$

Given,

Depth of Rain fall $= 65\ cm$

$\therefore$ percentage of rainfall has become runoff $= \dfrac{27}{6} \times 100$

$= 41.61\ \%$

Area required to irrigate crop $= \dfrac{\text{Volume}}{\text{Depth}} = \dfrac{5.68 \times 10^7}{0.6}$

$= 9.467 \times 10^7\ m^2$

$= 9.467 \times 10^3$ ha.

(Ans.)

Sheet-12

(2015) The mean daily flows at a gauging station for 7 days 7, 27, 58, 41, 31, 20 and 13 m^3/sec. respectively. What is the total volume of stream flow in hectare-meters? If the drainage area at the site is 100 km^2. what is the runoff volume?

solution: The total stream flow $= (7+27+58+41+31+20+13)\ m^3$

$= 197\ m^3/sec$

$= 197 \times 60 \times 60 \times 24\ m^3/days.$

$= 17020800\ m^3/days.$

$= 1702.08$ ha-m $\quad [10^4\ m^3 = 1$ ha-m$]$

Runoff volume in cm

$= \dfrac{1702080\ m^3}{100 \times 10^6\ m^2}$

$= 0.1702\ m = 17.02\ cm$ (Ans.)

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Hydrograph

Example - 5.13 (Raghunath) - Page - 162

DRO peak = Flood peak − BF

= (150 − 6) cumec.

= 144 cumec

$$P_{net} = \frac{\text{DRO peak}}{\text{UG peak}} = \frac{144}{36} = 4 \text{ cm}$$

∴ Depth of the storm rainfall, P = Pnet + losses

$$= 4 + \frac{6}{10} \times 6$$

$$= (4 + 3.6) \text{ cm}$$

$$= 7.6 \text{ cm}$$

We know,

DRO = UGO × Pnet

Again, TRO = DRO + BF

= UGO × Pnet + BF

(I) TRO = 0 × 4 + 6 = 6

(II) TRO = 15 × 4 + 6 = 66

Similarly, the stream flow ordinates are 150, 126, 76, 40, 18, 6 cumec. respectively.

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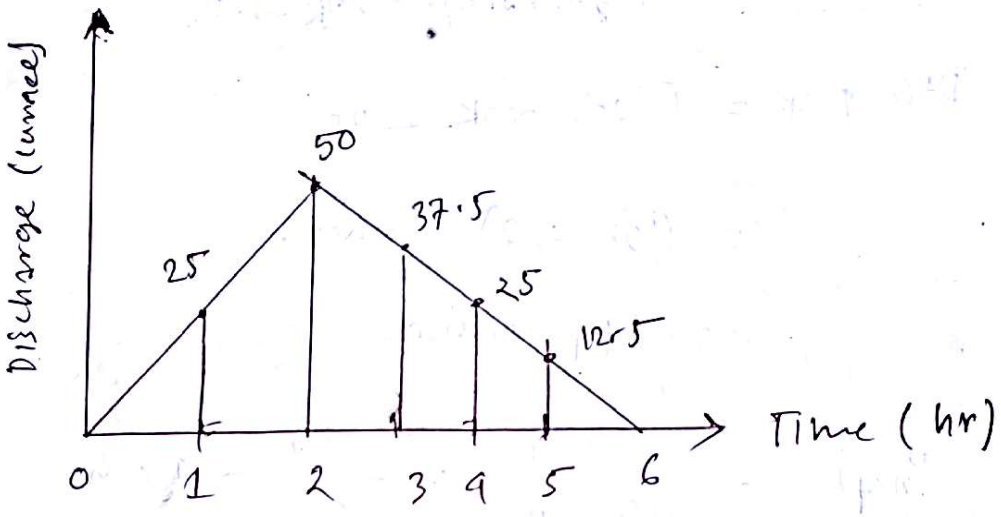
Example: 5.11 (Raghunath) - Page - 160 (2015, 2013, 2005)

Given,

Loss = 9 mm/hr = 0.9 cm/hr

B.F = 10 cumec



(i)

Time (hr)	UGO (cumec)	DRO due to rainfall excess (cumec)		Total DRO (cumec)	BF (cumec)	TRO (cumec)	Remark
		(4.9 − 0.9) X UGO = 1 UGO	Prct X UGO = (3.9 − 0.9) X UGO = 3 UGO				
0	0	0	–	0	10	10	
1	25	100	0	100	10	110	
2	50	200	75	275	10	285	
3	37.5	150	150	300	10	310	– Peak
4	25	100	112.5	212.5	10	222.5	
5	12.5	50	75	125	10	135	
6	12.5	0	37.5	37.5	10	47.5	
7	–	–	0	0	10	10	

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(ii) Volume of water one basin = Area of UG (triangle)

$$(A \times 10^6) \times \frac{1}{100} = \frac{1}{2}(6 \times 60 \times 60) \times 50$$

$$\Rightarrow A = 54 \text{ km}^2$$

(iii) co-efficient of run off, $C = \frac{R}{P} = \frac{(4.9 - 0.9) + (3.9 - 0.9)}{4.9 + 3.9}$

$$= 0.795$$

(Ans.)

Sheet – 30

(2012) (2014)

(3hr to 9hr)

Time hr	3 hr UGO (curve)	3hr UGO (lagged 3hr)	3hr (UGO) (lagged 6hr)	Total ② + ③ + ④	9hr UGO ⑤ ÷ 3
1	2	3	4	5	6
0	0	–	–	0	0
3	50	0	–	50	16.67
6	75	50	0	125	41.67
9	150	75	50	275	91.67
12	80	150	75	305	101.67
15	50	80	150	280	93.33
18	30	50	80	160	53.33
21	0	30	50	80	26.67
24	–	0	30	30	10
27	–	–	0	0	0

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Stream Gauging

Example-6.3 (Raghunath) — 185 page.

$\Delta h_0 = 12.00 - 11.65 = 0.35\ m$

$\Delta h_a = 12.00 - 11.02 = 0.98\ m$

We know, $\left(\frac{Q_a}{Q_0}\right) = \left(\frac{\Delta h_a}{\Delta h_0}\right)^n$

$\Rightarrow \frac{15.20}{9.5} = \left(\frac{0.98}{0.35}\right)^n$

$\Rightarrow \log(1.6) = n \log(2.8)$

$\Rightarrow n = 0.456$

Now, when auxiliary gauge reads 11.37 m

$\Delta h_a = (12 - 11.37) = 0.63\ m$

$\therefore \frac{Q_a}{9.5} = \left(\frac{0.63}{0.35}\right)^{0.456}$

$\therefore Q_a = 12.42$ cumec.

(Ans.)

Question-3(b) (Raghunath) — 189 page

Sheet-25

(2010, 06, 12, 13)

We know, $Q = AV$

$= (bd) \times V$

$= (1 \times 5) \times V$

$\therefore Q = 5V$

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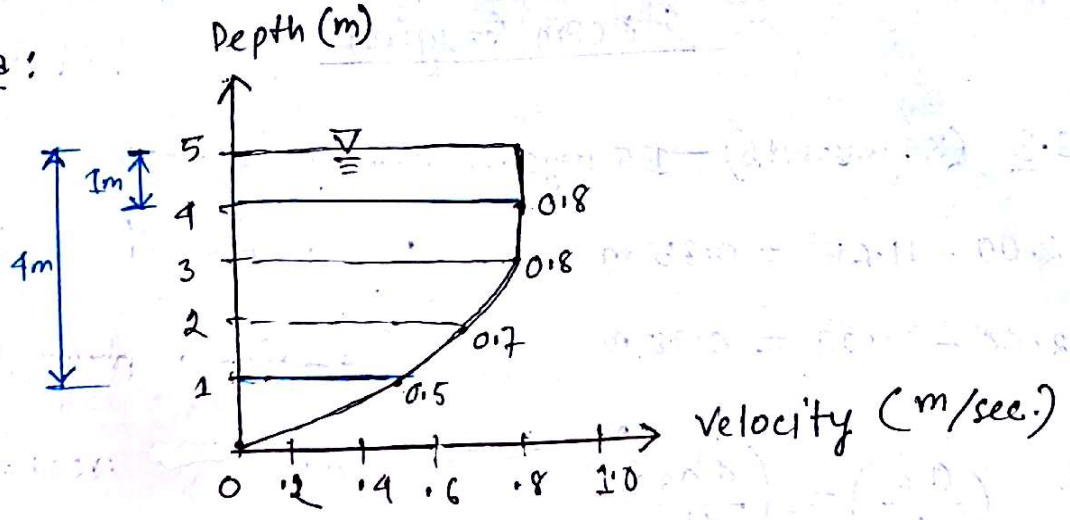
From data:



For two point method,

$$V = \frac{V_{0.2d} + V_{0.8d}}{2}$$

From top

Here, $0.2d = 0.2 \times 5 = 1\,m$, then $V = 0.8\ m/s$

$0.8d = 0.8 \times 5 = 4\,m$, then, $V = 0.5\ m/s$

$$\therefore V = \frac{0.8 + 0.5}{2}\ m/s$$

$$\Rightarrow V = 0.65\ m/s$$

$$\therefore Q = 5V = (5 \times 0.65) = 3.25\ m^3/sec.$$

(Ans.)

Question-9 (Raghunath) – Page 190 (2014) sheet- 24

$$\text{surface velocity} = \frac{L}{t} = \frac{20}{10} = 2\ m/sec.$$

We know, $V_{surface} = \frac{V_{mean}}{0.85}$

$$\therefore V_{mean} = 0.85 \times V_{surface} = (0.85 \times 2)\ m/sec.$$

$$\therefore V_{mean} = 1.7\ m/sec$$

∴ For velocity rod, $V = 1.7\ m/sec.$ $L = 20\,m$ (same run)

$$\therefore t = \frac{L}{V} = \frac{20}{1.7} = 11.76\ m/sec.$$

(Ans.)

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Example – 6.1 (Raghunath) – Page 179

Distance from one end of water surface (m)	Depth, d (m)	Immersion of current meter below water surface: depth = x_d (x = 0.6, 0.2, 0.8) (m)	Rev. R	Time (sec)	$N = \frac{R}{t}$ (rps)	$V = aN + b$, a = 0.3, b = 0.05 (m/sec)	Average velocity in strip V (m/sec)	Discharge $Q = AV = (bd)V$ (cumec)
3	1.9	(.6 × 1.9) = 1.14	12	50	0.24	0.122	0.122	0.51
6	3.3	(.2 × 3.3) = 0.66	38	52	0.73	0.269	$\frac{(0.269+0.176)}{2}$ = 0.223	2.16
		(.8 × 3.3) = 2.64	23	55	0.42	0.176		
9	5.0	(.2 × 5.0) = 1.0	40	58	0.69	0.257	0.236	3.54
		(.8 × 5) = 4.0	30	54	0.56	0.218		
12	9.0	(.2 × 9) = 1.8	48	60	0.80	0.290	0.259	7.0
		(.6 × 9) = 7.2	34	58	0.59	0.227		
15	5.4	(.2 × 5.4) = 1.08	34	52	0.65	0.245	0.238	3.85
		(.8 × 5.4) = 4.32	30	50	0.60	0.230		
18	3.8	(.2 × 3.8) = 0.76	35	52	0.67	0.251	0.234	2.68
		(.8 × 3.8) = 3.04	30	54	0.56	0.218		
21	1.8	(.6 × 1.8) = 1.08	18	50	0.36	0.158	0.158	0.86

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Example–7.4 (Suresh)

Find the mean stream flow velocity for $a = 0.4$ and $b = 0.06$ using data.

Stream depth :	0.2d	0.6 d	0.8d
Revolution :	40	15	30
Time (sec) :	58	50	54

Solution:

We know, $V = aN + b = 0.4N + 0.06$

$\therefore V_{0.2d} = 0.4 \times \left(\frac{40}{58}\right) + 0.06 = 0.34$ m/sec

$V_{0.6d} = 0.4 \times \left(\frac{15}{50}\right) + 0.06 = 0.18$ m/sec.

$V_{0.8d} = 0.4 \times \left(\frac{30}{54}\right) + 0.06 = 0.28$ m/sec.

$\therefore V_{mean} = \frac{0.34 + 0.18 + 0.28}{3} = 0.27$ m/sec.

(Ans.)

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Example-7.5 (Suresh) (2008) Sheet-28

Compute the value of stream flow rate using two point method.

Depth from end (m)		0	1	2	3	4	5	6	7
Depth (m)		0	1.4	3.3	5.0	9.0	5.4	3.8	1.8
velocity at m/sec.	0.2d	0	0.4	0.6	0.84	0.90	0.80	0.62	0.54
	0.6d	0	0.25	0.35	0.60	0.70	0.65	0.50	0.36
	0.8d	0	0.20	0.30	0.50	0.62	0.55	0.40	0.30

Solution:

By two point method,

$$Q = A V_m$$

$$= (bd) \times V_m \quad \text{where,} \quad V_m = \frac{V_{0.2d} + V_{0.8d}}{2}$$

Distance (m)	0	1	2	3	4	5	6	7
Depth, d (m)	0	1.4	3.3	5.0	9.0	5.4	3.8	1.8
$V_m = \frac{V_{0.2d} + V_{0.8d}}{2}$	0	$\frac{0.4+0.20}{2} = 0.3$	$\frac{0.6+0.3}{2} = 0.45$	0.67	0.76	0.68	0.51	0.42
$Q = (bd) \times V_m$ (cumec)	0	0.42	1.49	3.35	6.84	3.67	1.94	0.76

∴ Total flow rate $= (0.42 + 1.49 + 3.35 + 6.84 + 3.67 + 1.94 + 0.76)$

$= 18.47$ cumec.

Ans.)

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Design Flood & Flood Routing

Sheet – 54

(2012) Given, $N = 10$ years.

$T_r = 5$ years. $\therefore p = \frac{1}{5} = 0.2$

(i) probability of not occuring flood $= (1-p)^N = (1-0.2)^{10}$

$= 0.1074$

$= 10.74\%$

(ii) probability of occuring flood twice $(r=2) = {}^{n}C_r \, p^r \cdot q^{n-r}$

$= {}^{10}C_2 (0.2)^2 \cdot (1-0.2)^{10-2}$

$= 0.302$

$= 30.2\%$ (Ans)

Example – 19.6 (Reddi) Page-497

(2010)

For $n = 35$ years, $\bar{y}_n = 0.54034$

$\sigma_n = 1.12847$

For $T = 50$, $y_T = -\ln\left[\ln\left(\frac{T_r}{T_r - 1}\right)\right]$

$y_{50} = -\ln\left[\ln\left(\frac{50}{50-1}\right)\right] = 3.902$

$K_{50} = \frac{y_T - \bar{y}_n}{\sigma_n} = \frac{3.902 - 0.54034}{1.12847} = 2.9789$

For $T = 100$

$y_{100} = -\ln\left[\ln\left(\frac{100}{100-1}\right)\right] = 4.60015$

$K_{100} = \frac{4.600015 - 0.54034}{1.12847} = 3.59762$

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We know, $x_T = \bar{x} + K_{50} S_x$

Given

$x_{50} = 660$ and $x_{100} = 740$

$\therefore 660 = \bar{x} + 2.9789 S_x$ ..... (I)

and $740 = \bar{x} + 3.59762 \cdot S_x$ ..... (II)

From (I) & (II) we obtain,

$\bar{x} = 274.83 \ m^3/sec.$

$S_x = 129.3 \ m^3/sec.$

Now, $y_{200} = -\ln\left[\ln\left(\frac{200}{200-1}\right)\right] = 5.29581$

$K_{200} = \frac{5.29581 - 0.57934}{1.12847} = 4.2141$

$\therefore x_{200} = \bar{x} + K_{200} \cdot S_x$

$= 274.83 + 4.2141 \times 129.3$

$= 819.71 \ m^3/sec.$

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Example-14.8 (Reddi) - Page- 501

Given, $T_r = 25$

$N = 5$

$R = \frac{1}{T_r} \Rightarrow \frac{1}{25} = 0.04$

The risk of future, $R = 1 - (1-P)^N$

$= 1 - (1 - 0.04)^5$

$= 0.1846$

$= 18.46\%$

if risk is to be reduced to 10%, $\therefore R = \frac{10}{100} = 0.1$

$R = 1 - (1-P)^N$

$\Rightarrow 0.1 = 1 - (1-P)^5$

$\Rightarrow P = 0.02085$

$\therefore T = \frac{1}{0.02085}$ years $= 47.96$ years.

(Ans.)

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Example-14.9 - (Reddi) - Page 501

Sheet - 59, 60

(2013, 2015)

Given, Assurance = 95% = 0.95

N = 50 year.

We know, probability to not occuring failure = $(1-P)^N$

$\therefore (1-P)^N = 0.95$

$\Rightarrow (1-P)^{50} = 0.95$

$\Rightarrow P = 1.025 \times 10^{-3}$

$\therefore T_r = 975.286$ years.

For n = 30 years,

Given, $\bar{y}_n = 0.53622$ and $\sigma_n = 1.11238$

$\therefore y_T = -\ln\left[\ln\left(\frac{975.286}{975.286-1}\right)\right] = 6.88223$

$K_T = \frac{6.88223 - 0.53622}{1.11238} = 5.705$

$\therefore$ Design flood, $x_T = \bar{x} + K_T \cdot S_n$

Given, $\bar{x} = 1200$ m3/s

$S_n = 650$ m3/s

$= 1200 + 5.705 \times 650$

$= 4908.25$ m3/sec.

(Ans)

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Example – 14.5 (Reddi) – Page-496

Step-1:

$$\bar{x} = \frac{395 + 619 + 766 + \cdots + 610}{40} = 530 \text{ m}^3/\text{s}$$

$$S_x^2 = \frac{(395-530)^2 + (619-530)^2 + \cdots + (610-530)^2}{40-1} = 26097.436$$

$\therefore S_x = 161.55 \text{ m}^3/\text{sec}.$

Step-2:

Step-3: For $n = 40$, Given $\bar{y}_n = 0.54362$ and $\sigma_n = 1.14132$

For $T_r = 100$ years, & $T_r = 20$ years.

$$\therefore y_{100} = -\ln\left[\ln\left(\frac{100}{100-1}\right)\right] = 4.6$$

$$y_{200} = -\ln\left[\ln\left(\frac{200}{200-1}\right)\right] = 5.2928$$

Step-4:

$$K_{100} = \frac{4.6 - 0.54362}{1.14132} = 3.554$$

$$K_{200} = \frac{5.2958 - 0.54362}{1.14132} = 4.164$$

Step-5:

$$x_T = \bar{x} + K_T \cdot S_x$$

$$x_{100} = 530 + 3.554 \times 161.547 = 1104.14 \text{ m}^3/\text{s}$$

$$x_{200} = 530 + 4.164 \times 161.547 = 1202.68 \text{ m}^3/\text{s}$$

(Ans.)

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Sheet-56

(2009)

Given,

$P = 5\% = 0.05$

$N = 120$ days

$\lambda = PN = (0.05 \times 120) = 6$

Probability that the crop will fail in a year,

$$P(r=1) = \frac{e^{-\lambda} \lambda^r}{r!}$$

$$= \frac{e^{-6} 6^1}{1!} = 0.0149$$

$\therefore P(r=1) = 1.49\%$

Probability that the crop will fail in 4 years,

$$P(r=4) = \frac{e^{-6} 6^4}{4!} = 0.134$$

$P(r=4) = 13.4\%$

(Ans)

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Example – 13.4 (Reddi) 461 (2007)

$K = 12, \quad x = 0.278$

$\Delta t = 4,$

$$\frac{\Delta t}{K} = \frac{4}{12} = \frac{1}{3}$$

$$C_0 = \frac{\left(\frac{\Delta t}{K}\right) - 2x}{2(1-x) + \left(\frac{\Delta t}{K}\right)} = \frac{\frac{1}{3} - 2 \times 0.278}{2(1-0.278) + \frac{1}{3}} = -0.125$$

$$C_1 = \frac{\left(\frac{\Delta t}{K}\right) + 2x}{2(1-x) + \left(\frac{\Delta t}{K}\right)} = \frac{\frac{1}{3} + 2 \times 0.278}{2(1-0.278) + \frac{1}{3}} = 0.50$$

$$C_2 = \frac{2(1-x) - \left(\frac{\Delta t}{K}\right)}{2(1-x) + \left(\frac{\Delta t}{K}\right)} = \frac{2 \times (1-0.278) - \frac{1}{3}}{2(1-0.278) + \frac{1}{3}} = 0.625$$

Check: $C_0 + C_1 + C_2 = -0.125 + 0.5 + 0.625 = 1.0$

Now, the routing equation is,

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$$

$$\Rightarrow Q_2 = -0.125 I_2 + 0.5 I_1 + 0.625 Q_1$$

For time interval, (0–4) h, $I_2 = 68 m^3/s$ & $Q_1 = I_1 = 42$ $\therefore Q_2 = -.125 \times 68 + .5 \times 42 + 0.625 \times 42$

$= 38.75 m^3/s$

Then for (4–8) h, $Q_1 = Q_2 = 38.75$

and $I_1 = 68 m^3/s$, $I_2 = 116 m^3/s$

Similarly ---------- (Table)

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Channel Routing by Muskingum Method

Time h	Inflow I	$-0.125 I_2$	$0.5 I_1$	$0.625 Q_1$	Outflow Q
0	42				42
4	68	-8.5	21	26.25	38.75
8	116	-14.5	34	24.22	43.72
12	164	-20.5	58	27.33	64.83
16	174	-24.25	82	40.52	98.27
20	200	-25.0	97	62.42	133.42
24	192	-24.0	100	83.39	159.39
28	170	-21.25	96	99.62	174.37
32	150	-18.75	85	108.98	175.23
36	128	-16.0	75	109.52	168.52
40	106	-13.25	64	105.33	156.08
44	88	-11.00	53	97.55	139.55
48	74	-9.25	44	87.22	121.97
52	62	-7.75	37	76.33	105.58
56	54	-6.75	31	65.93	90.18

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9

Groundwater

9.1 INTRODUCTION

In general, the term *groundwater* or *subsurface water* refers to the water that occurs below the surface of earth. The main source of groundwater is infiltration. The infiltrated water after meeting the soil moisture deficiency percolates deeply and becomes groundwater. The groundwater is free from pollution and is very useful for domestic use in small towns and isolated farms. It can be made available at a small capital cost and also in least possible time. In arid regions, groundwater is often the only reliable source of water for irrigation. Groundwater has been an important water resource throughout the ages. Like any other natural resource, groundwater is also not unlimited. They must be wisely managed and protected against undue exploitation and contamination by pollutants or salt water. The subject of groundwater attracted the attention of agricultural engineers, civil engineers, geologists, geophysicists and scientists from various other desciplines. Excellent books are available on the subject. This chapter presents an elementary treatment of groundwater as a part of the hydrologic cycle.

The formation below the earth's surface is divided into two zones by an irregular surface called the *water table*. At all points on the water table the pressure is atmospheric. The zone between the ground surface and the water table is called the *unsaturated zone* or the *vadose zone*. In the vadose zone the soil pores may contain either air or water or both. Hence it is also called the zone of aeration. (vide Fig 9.1). Water in this zone is held to the soil particles by capillary forces. Thus, while this water is able to move within the vadose zone, it cannot move out of the zone into wells or other places that are exposed to atmospheric pressure. In the zone below the water table all the soil pores are completely filled with water and hence it is called the *zone of saturation* or the *phreatic zone*. The phreatic zone may extend to considerable depth, but as the depth increases the weight

of overburden tends to close the pore space and relatively little water is found at depths greater than 3 km. Since the water in the phreatic zone is under pressure more than atmospheric, if a hole is dug in this zone water begins to flow into the hole whereas it is not so in the vadose zone. It is this water in the phreatic zone which moves freely under the force of gravity that is actually called the groundwater, the occurrence and movement of which are the topics for discussion in this chapter. The water in the vadose zone is of interest to agronomists and agricultural engineers.

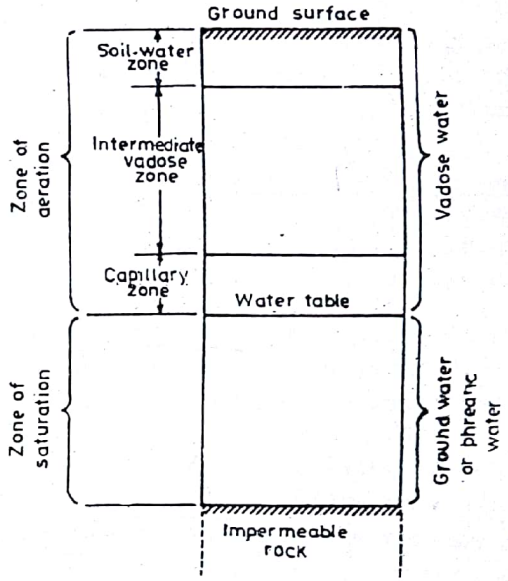

Fig 9.1. Definition sketch of zones of saturation and aeration.

9.2 OCCURRENCE OF GROUNDWATER

Groundwater occurs at various locations below the earth's surface depending on the physical properties of various formations that exist. Fig. 9.2 is a typical cross-section of the earth's crust showing the occurrence of grounwater.

Aquifer. Formations which contain groundwater and at the same time which are sufficiently permeable to transmit and yield water in usable quantities are called the aquifers. They may get recharged directly from above through the processes of precipitation and infiltration, or they may have

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the recharge area somewhere else on the earth's surface. The amount of water contained by the aquifer depends on the *porosity* of the aquifer formation while the amount of water that it can yield depends on its *permeability.*

Unconfined Aquifer. An aquifer having water table in it is called an unconfined aquifer. For this aquifer water table serves as the upper surface of zone of saturation while a less permeable or essentially impermeable layer defines its lower boundary. The impermeale layer underlying an unconfined aquifer may be made of clay, shale, solid limestone, igneous rock or bed rock. Unconfined aquifer is also known as a *free aquifer, phreatic aquifer, water table aquifer,* or *nonartesian aquifer.* The water table is an unconfined aquifer varies in undulating form and in slope, depending upon the areas of recharge and discharge, pumpage from wells and its permeability. A well driven into an unconfined aquifer will indicate a static water level corresponding to the water table level at that location.

Confined Aquifer. When an aquifer is sandwiched between two layers of much less permeable material then it is called a confined aquifer. For example, a sandy layer between two clay layers, sandstone layer between two layers of shale etc. It is also known as a *pressure aquifer*, or an *artesian aquifer*. Confined aquifers are completely filled with water and they do not have a free water table. They will have recharge area somewhere on the earth surface. The water in confined aquifers will be under pressure and the pressure condition in the aquifer is characterised by the *piezometric surface,* which is the surface obtained by connecting equilibrium water levels in the tubes or *piezometers* penetrating the confined aquifer (Fig 9.2). A horizontal surface in level with the water table at the recharging area may be taken as the approximate piezometric surface of a confined aquifer. If a well penetrates into the confined aquifer, water level in the well will rise to piezometric level. If the piezometric level at the place of the well is above the upper confining layer the static water level in the well will be above the aquifer. Such a well is called an *artesian well.* If the piezometric surface at the place of the well is above the ground level, the confined aquifer will yield a *free-flowing well* also known simply as a *flowing well.* A well penetrating into an unconfined aquifer, on the other hand, is called a *water table well.* The groundwater can also move to ground surface through natural passages like *faults* or *sinkholes*, where it then produces a *spring.*

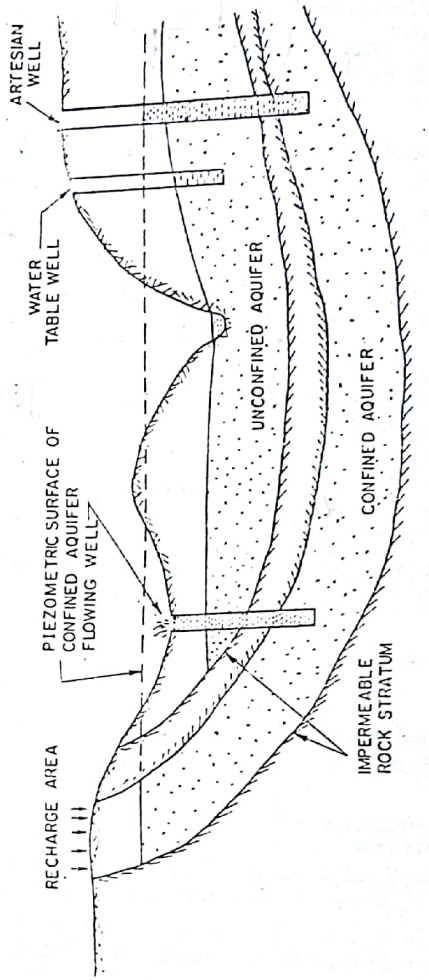

Fig 9.2. Groundwater in confined and unconfined aquifers.

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Aquiclude. A geological formation which is saturated and which may contain large amount of water because of its high porosity but cannot transmit water as it is relatively impermeable is called an aquiclude. A clay layer is an example of an aquiclude.

Aquitard. It is a saturated geological formation which is poorly permeable and hence it does not yield water freely to wells. It may transmit vertically appreciable quantities of water to or from adjacent aquifers. Sandy clay is an example of aquitard.

Aquifuge. An impervious geological formation which neither contains nor transmits water is known as an aquifuge. Solid granite rock is an example of aquifuge.

Leaky Aquifer. An aquifer bound by one or two aquitards is called a leaky aquifer or a *semi-confined* aquifer.

Perched groundwater. When a groundwater body is separated from the main groundwater by a relatively impermeable stratum of small areal extent and by the zone of aeration above the main body of groundwater, it is called perched groundwater (vide Fig 9.3). Wells tapping groundwater from these sources yield only temporarily or small quantities of water.

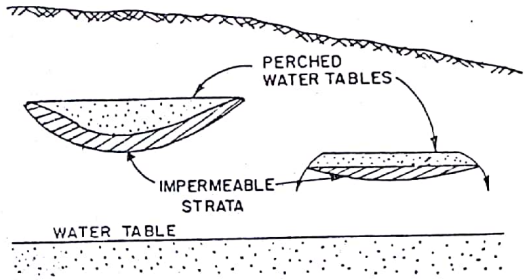

Fig 9.3. Perched water table.

Influent and Effluent Streams. If the water table of an unconfined aquifer intersects a surface stream, then aquifer contributes water to the stream. Such a stream is called an effluent stream. If a surface stream runs well above the water table, water flows from the stream to the aquifer. Such a stream is called an influent stream (vide Fig 9.4). Most of the flow of *perennial* streams in non-rainy periods originate from groundwater when they behave as effluent streams. Likewise a large part of the flow of *ephemeral streams* (which go dry during prolonged rainless period) may percolate and become groundwater.

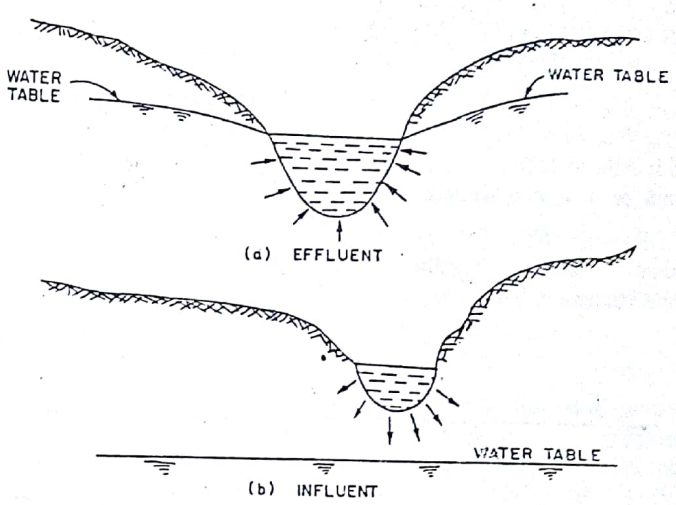

Fig. 9.4. Effluent and influent streams.

9.3 AQUIFER PARAMETERS

An aquifer has the ability to store and transmit water. The quantity of water stored by an aquifer and the quantity of water released by it depend on the nature and composition of the aquifer which are quantified through certain parameters like *porosity, specific yield, storage coefficient, permeability* and *transmissivity*. These aquifer parameters are now briefly discussed.

Porosity. The ratio of volume of pore space, V_v to the volume of the formation V is called the porosity, usually denoted by n. Thus we have

$$n = \frac{V_v}{V} \qquad \ldots (9.1)$$

In unconsolidated materials, the porosity depends on the shape and size distribution of the particles and their packing arrangement. The *original porosity* of a material is that which existed at the time the material was formed. *Secondary porosity* results from fractures and solution channels.

Original porosity can be measured in the laboratory using an undisturbed sample. The sample is ovendried and its weight w_1 measured. It is then saturated with some liquid (the liquid may have to be forced into the sample under pressure to ensure complete saturation) and its weight w_2 is again measured. Finally the saturated sample is immersed in a vessel containing the same liquid and the weight of the liquid displaced by the sample, w_3 is noted. The weight of liquid required to saturate the sample,

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that is $(w_2 - w_1)$, divided by the weight of the liquid displaced by the saturated sample, that is w_3, is the porosity of the sample. Multiplication of this ratio with 100 gives the porosity by percentage. Secondary porosity cannot be measured without an impossibly large sample.

Specific Yield. The specific yield of an aquifer is the ratio of the volume of water which will drain freely from the material to the volume of the formation. Therefore,

$$S_y = \frac{w_y}{V} \qquad \ldots (9.2)$$

where w_y is the volume of water drained and S_y is the specific yield. Values of specific yield depend on grain size, shape and distribution of pores and compaction of the formation. Specific yield of fine-grained aquifers will be small, while coarse-grained material will have high specific yield. When the material is drained, some water will always be retained in the small pore spaces under capillary tension and therefore the volume of water drained will never be equal to the volume of pore spaces. That is $w_y < V_v$. Hence the specific yield is always less than porosity.

Specific Retention. The specific retention S_r is defined as the ratio of the volume of water retained w_r in the material to the total volume of the material, when a saturated material is dewatered. Thus

$$S_r = \frac{w_r}{V} \qquad \ldots (9.3)$$

Since w_r and w_y together constitute the total volume of water which is the total pore space, it is obvious that the specific retention and the specific yield are complimentary quantities whose sum represents the porosity of the aquifer. That means,

$$n = S_r + S_y \qquad \ldots (9.4)$$

Storage coefficient. The storage coefficient S is defined as the volume of water that an aquifer releases from or takes into storage per unit surface area of the aquifer per unit drop of water table in the case of an unconfined aquifer and per unit drop of piezometric surface in the case of a confined aquifer. It is also known as *storativity*. The concept of storage coefficient is better illustrated through Fig 9.5.

As depicted in Fig 9.5 *(b)*, when there is a change in water table water physically enters or leaves the unconfined aquifer. Therefore for this case the storage coefficient equals the specific yield. Fig 9.5 *(a)* shows the situation in a confined aquifer. When there is a change in the piezometric surface it does not represent the water released from the aquifer because a confined aquifer remains completely saturated at all times. The volume

of water released from or taken into storage in this case is because of the elastic response of the aquifer, confining beds and water to the pressure changes.

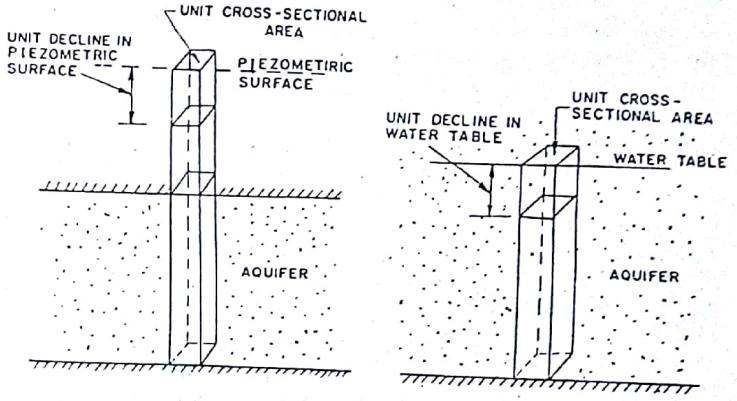

Fig. 9.5. Definition sketch of storage coefficient.

The storage coefficient of a confined aquifer will be very small in the range of 0.00005 to 0.005, indicating that large pressure changes over extensive areas are required to produce substantial water yields. The storage coefficient of unconfined aquifer on the other hand is usually large and of the order 0.01 to 0.3.

Permeability. The permeability of a material is a measure of its capacity to transmit water or any other fluid through its interstices. Groundwater is transmitted through aquifers at very small velocities ranging from 1 to 500 m/year. The laminar flow principles are shown to be applicable for the flow of groundwater in aquifers by Darcy. The *Darcy's law* states that the rate of flow per unit area of an aquifer is proportional to the gradient of the potential head measured in the direction of flow. Since the velocities are so small that the velocity heads are negligible the gradient of the potential head equals the hydraulic gradient. And since the flow rate per unit area denotes the velocity of flow, the Darcy's law may be written as

$$v = Ki = -K \frac{dh}{dl} \qquad \ldots (9.5)$$

where the constant of proportionality K is known as the *hydraulic conductivity* or the *coefficient of permeability*, v is the velocity of water through porus medium, and i or dh/dl is the hydraulic gradient. The negative sign indicates that the flow takes place in the direction of falling head.

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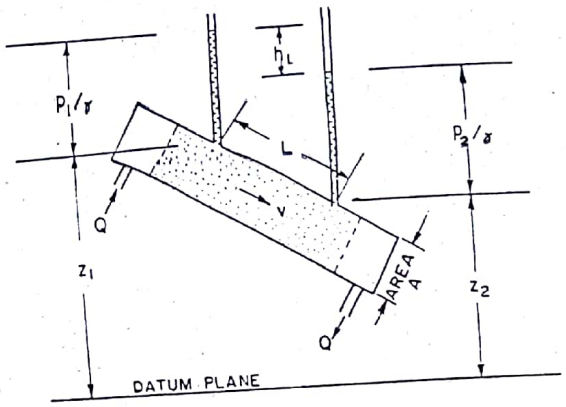

Fig 9.6 Head loss in flow through porus media.

The verification of Darcy's law follows from a simple experiment shown in Fig 9.6. Applying Bernoulli energy equation for the two sections where piezometers are inserted

$$\frac{p_1}{\gamma} + z_1 + \frac{v_1^2}{2g} = \frac{p_2}{\gamma} + z_2 + \frac{v_2^2}{2g} + h_L \qquad \text{... (9.6)}$$

where p is pressure, γ is the specific weight of water, v is the velocity of flow, g is the acceleration due to gravity, z is the elevation and h_L is the head loss. Subscripts 1 and 2 refer to the two sections separated by a distance L. Neglecting the velocity heads as they will be very very small compared to other terms and on rearrangement we obtain

$$h_L = \left(\frac{p_1}{\gamma} + z_1\right) - \left(\frac{p_2}{\gamma} + z_2\right) \qquad \text{... (9.7)}$$

which shows that the head loss is independent of the inclination of the cylinder. Darcy's experiments showed that the discharge Q is directly proportional to head loss h_L and area of flow, and inversely proportional to the length L. That is

$$Q \propto \frac{A}{L}.h_L \; ; \quad \text{or} \quad \frac{Q}{A} \propto \frac{h_L}{L} \; ; \quad \text{or} \quad v \propto \frac{h_L}{L}.$$

In the limiting case as the two sections become closer and closer, $\frac{h_L}{L}$ tends to $\frac{dh}{dl}$. Introducing a constant of proportionality, $v = -K\frac{dh}{dl}$; which is the Darcy's law. In Eq. (9.5), if $\frac{dh}{dl}$ equals unity v equals K. Hence the hydraulic conductivity may also be defined as the rate of flow per unit area of an aquifer under a unit hydraulic gradient. Since hydraulic gradient is

dimensionless, if follows from Eq. (9.5) that the hydraulic conductivity has the same units as velocity and can be expressed in cm/s or m/s. But when hydraulic conductivity of aquifers is expressed in these units the value will be very small and therefore it is usually expressed in m/day.

The velocity v in Eq. (9.5) is variously known as the *Darcy's velocity, discharge velocity*, or *apparent velocity*. It is based on the assumption that the flow occurs through the entire cross-section of the material without regard to solids and pores. Actually the flow is limited to pore space only. Thus if v_a is the actual velocity, one can write

$$Q = A.v = A_p.v_a$$

where A_p is the area of the pore space. Hence

$$v_a = v.\frac{A}{A_p} = \frac{v}{(A_p/A)}.$$

But the ratio A_p/A may be taken to be approximately equal to the porosity of the material and can be replaced by n. Therefore, we have

$$v_a = \frac{v}{n} \qquad \text{... (9.8)}$$

Since the pore space vary continuously with location within the medium, the actual velocity is non-uniform and varies from point to point.

The Darcy's law is found to be valid for the flows with *Reynold's number* less than unity . The Reynold's number R_e is given by

$$R_e = \frac{\rho v d_{10}}{\mu} \qquad \text{... (9.9)}$$

where ρ and μ are the density and viscosity of the fluid, v is the Darcy's velocity and d_{10} is an effective grain size. The Darcy's law does not depart seriously upto $R_e < 10$ which may be taken as the upper limit. Most groundwater flows occur with $R_e < 1$ and so Darcy's law is applicable. Deviations from Darcy's law can occur where steep hydraulic gradients exist, such as near pumped wells and in formations with large underground openings.

The hydraulic conductivity is dependent on both the properties of the fluid and the properties of the medium. It can be proved that K is given by

$$K = \frac{cd^2\rho g}{\mu} \qquad \text{... (9.10)}$$

where ρ and μ are the density and viscosity of the fluid, g is the acceleration due to gravity, d is the average pore size of the medium and c is a factor accounting for the shape, packing and other characteristics of the material in the medium. The *intrinsic permeability* which is the property of the medium only and which does not depend on the fluid properties is given by

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$$k = \frac{K\mu}{\rho g} = cd^2 \qquad \text{... (9.11)}$$

where k is the intrinsic permeability, and K is the hydraulic conductivity for the given fluid with properties μ and ρ. It can be seen that k has the units of area. The customary unit of k is the *darcy* which is equal to 0.987×10^{-12} m^2. The intrinsic permeability is not used in groundwater studies. It is mainly applied in the petroleum and natural gas industries which deal with underground flow of fluids of different viscosity and density including liquids and gases.

The hydraulic conductivity can be measured in a laboratory using *permeameters*. The values obtained through laboratory tests are not truly representative of the actual hydraulic conductivity in the field. This is because undisturbed samples of unconsolidated materials are difficult to obtain, while disturbed samples experience changes in porosity, packing and grain orientation. Also, flow in solution cavities or rock fractures, and the effect of large boulders in gravel aquifers cannot be duplicated in a laboratory sample. The average hydraulic conductivity of an aquifer for a large area around the test well can be determined from pumping tests using the principles of well hydraulics that are discussed later in the chapter.

Table 9.1 gives the approximate average values of porosity, specific yield and permeability of some typical materials.

Table 9.1. Approximate average porosity, specific yield and permeability of various materials

Material	*Porosity %*	*Specific yield %*	*Hydraulic conductivity m/day*	*Intrinsic permeability darcys*
Clay	45	3	0.0004	0.0005
Sand	35	25	41.0	50
Gravel	25	22	4100.0	5000
Gravel and Sand	20	16	410.0	500
Sandstone	15	8	4.10	5
Dense limestone and shale	5	2	0.041	0.05
Quartzite, granite	1	0.5	0.0004	0.0005

Transmissivity. The transmissivity of an aquifer is also known as *transmissibility*. It is the product of the hydraulic conductivity K, and the thickness of the aquifer b and is expressed in units of m^2/day. If we consider a unit width of an aquifer of thickness b, the discharge through it is given by

$$Q = \text{Area} \times \text{Velocity} = (b \times 1) \times K\left(-\frac{dh}{dl}\right)$$

$$Q = Kb\left(-\frac{dh}{dl}\right) = T\left(-\frac{dh}{dl}\right) \qquad \text{... (9.12)}$$

where $T = Kb$ is the transmissivity. When the hydraulic gradient equals unity, the discharge is equal to T itself. In other words, the transmissivity of an aquifer may be defined as the discharge through the aquifer of one metre width under a unit hydraulic gradient. The value of T ranges from about 12 m^2/day to about 1200 m^2/day depending on the type and thickness of the aquifer. It is usually evaluated from the pumping tests.

Example 9.1. *An undisturbed rock sample has an oven dry weight of 0.655 kg. After saturation with kerosene its weight is 0.732 kg. It is then immersed in kerosene and found to displace 0.301 kg. What is the porosity of the sample ?*

Solution. Oven-dry weight $w_1 = 0.655$ kg

Weight of saturated sample $w_2 = 0.732$ kg

Weight of kerosene required to saturate the sample

$= (w_2 - w_1)$

$= 0.732 - 0.655 = 0.077$ kg

Weight of kerosene displaced by the saturated sample $w_3 = 0.301$ kg

Porosity of the sample $n = \dfrac{(w_2 - w_1)}{w_3}$

$= \dfrac{0.077}{0.301} = 0.2558$

$= 25.58\%$

Example 9.2. *When 3.68 million m^3 of water was pumped out from an unconfined aquifer of 6.2 km^2 areal extent, the water table was observed to go down by 2.6 m. What is the specific yield of the aquifer ? During a monsoon season if the water table of the same aquifer goes up by 10.8 m what is the volume of recharge ?*

Solution.

Water released from the aquifer $= (6.2 \times 10^6) \times 2.6 \times s_y$

Water pumped out $= 3.68 \times 10^6$ m^3

Equating these two quantities $s_y = \dfrac{3.68}{6.2 \times 2.6} = 0.2283$

$\therefore$ Specific yield of the aquifer $= 0.2283$ or 22.83%

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Volume of recharge $= 6.2 \times 10^6 \times 10.8 \times s_y$
$= 6.2 \times 10^6 \times 10.8 \times 0.2283$
$= 15.287$ million m^3.

Example 9.3. *The water table levels in two observation wells 350 m apart are + 210.5 and + 206.25 m respectively. If the hydraulic conductivity and porosity of the aquifer are 12.5 m/day and 15 per cent, what is the actual velocity of flow in the aquifer ?*

Solution. $v = K\left(-\frac{dh}{dl}\right)$

$$v = 12.5 \times \left[-\frac{(206.25 - 210.5)}{350}\right]$$
$$= 0.1518 \text{ m/day}$$
$$v_a = \frac{v}{n} = \frac{0.1518}{0.15} = 1.012 \text{ m/day}$$

$\therefore$ The actual velocity in the aquifer = 1.012 m/day.

Example 9.4. *A sample has a hydraulic conductivity of 10 m/day. What would be its intrinsic permeability ? What is its hydraulic conductivity in cm/s ? What would be its hydraulic conductivity at 30°C ?*

Solution. $k = \frac{K.\mu}{\rho g}$

$K = 10$ m/day $= 0.011574$ cm/s
$\mu = 0.01$ gm-cm/s
$\rho = 1$ gm/c.c.
$g = 981$ cm/s^2

$\therefore$ $k = \frac{0.011574 \times 0.01}{981} = 1.17982 \times 10^{-7}$ cm^2

But one darcy $= 0.987 \times 10^{-12}$ m^2 $= 0.987 \times 10^{-8}$ cm^2

$\therefore$ $k = \frac{1.17982 \times 10^{-7}}{0.987 \times 10^{-8}} = 11.954$ darcys

Intrinsic permeability of the sample = 11.954 darcys
Hydraulic conductivity in cm/s = 0.011574.

From Eq. (9.10), $K = \frac{cd^2 \rho g}{\mu}$

$$= \frac{cd^2 g}{(\mu/\rho)} = \frac{cd^2 g}{\nu}$$

where ν is the kinematic viscosity. Thus for a given pours medium the hydraulic conductivity is inversely proportional to the kinematic viscosity of the fluid. Therefore, we have

$$\frac{K}{K_t} = \frac{\nu_t}{\nu} \quad \text{or} \quad K_t = K \cdot \frac{\nu}{\nu_t}$$

where K_t and ν_t are the hydraulic conductivity and kinematic viscosity at some temperature t and K and ν are the hydraulic conductivity and kinematic viscosity at standard temperature of 20°C.

$$K_{30} = K \cdot \frac{\nu}{\nu_{30}} = 10 \times \frac{0.01}{0.008} = 12.5 \text{ m/day}$$

(Since kinematic viscosity for water at 20°C is 0.01 cm^2/s and at 30°C, it is 0.008 cm^2/s)

$\therefore$ Hydraulic conductivity of the sample at 30°C = 12.5 m/day.

9.4 GROUNDWATER MOVEMENT

Groundwater moves very slowly in the aquifers under differential head. A groundwater basin in filled and the excess water is discharged by several ways until a quasi-equilibrium is reached. All the perennial streams are generally effluent through at least a portion of their length gathering contributions from groundwater. When the water table is close to the surface groundwater may be discharged by direct evaporation and transpiration. Th groundwater movement in aquifers is governed by the established hydraul principles. The theory of groundwater movement is based on Darcy's law. The study of groundwater movement helps in predicting the yields from a well and the effects of future pumping, evaluating aquifer parameters, defining the aquifer boundaries, analysing the salt water intrusion problems etc. The theory of groundwater movement around wells is commonly termed the *well hydraulics*. There are two cases to be considered ; *steady state* and *unsteady state* flows. The steady state case rarely occurs in practice but the conditions after a prolonged pumping from a well may be approximated as steady state. The unsteady state case, or the *transient flow* case is of more practical significance. The following sections deal with steady and unsteady radial groundwater flow towards wells.

9.5 STEADY RADIAL FLOW TO WELLS

The analysis of groundwater movement will be greatly simplified if the vertical flow components are neglected and the groundwater is assumed to move primarily in a lateral direction. Accordingly the following assumption are made in the theory of groundwater movement.

1. The flow is horizontal and uniformly distributed in a vertical section.
2. The velocity of flow is proportional to the tangent of the hydraulic gradient instead of sine of the hydraulic gradient.

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The above two assumptions are called *Dupuit-Forchheimer* assumptions. In addition, the derivation of the steady flow discharge equation generally assumes the following.

3. The well is pumped at a constant rate.

4. The well fully penetrates the aquifer. (Equations can be derived for partially penetrating wells also. They are not discussed in this book).

5. The aquifer is homogeneous, isotropic, horizontal and of infinite horizontal extent.

6. Water is released from storage in the aquifer in immediate response to a drop in water table or piezometric surface.

Though the above assumptions are not strictly valid in all cases, they are required for making the analysis simpler and easier.

Steady Radial Flow to a Well in an Unconfined Aquifer. The flow system around a pumped well in an unconfined aquifer with an initially horizontal water table is depicted in Fig. 9.7. The thickness of aquifer is H and the well fully penetrates the aquifer.

If water is pumped at a constant rate from the well, a gradient in the water table towards the well is created which results in a depressing form of the water table. This is called the *cone of depression*. After a sufficiently long time when the contribution from the aquifer to the well is equal to the rate of pumping, the shape of the cone of depression remains fixed in space and will not change any further. When this condition is attained the flow is said to be steady. In practice this condition is rarely fulfilled and for the ease of analysis a steady state flow is assumed if the changes in water table elevation have become negligible with time. The decrease in the water level at the well is called the *drawdown*. If the drawdown is small compared to the thickness of the aquifer, the streamlines of the flow to the well may be assumed to be horizontal.

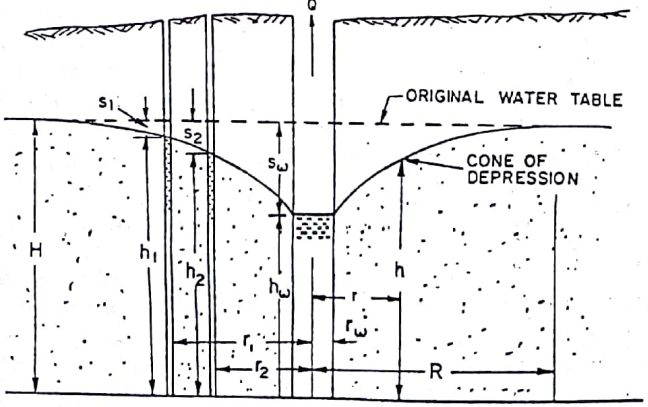

Fig. 9.7. Steady flow to a well in an unconfined aquifer.

In a steady state radial flow, as shown in Fig 9.7, the depth of flow at any radial distance r from the well is h, and the area of flow at this section is $2\pi rh$. According to Darcy's law the velocity of flow v is given by $V = K.i$, where i is the hydraulic gradient. Since the tangent of the water table is taken as hydraulic gradient, the velocity at this section becomes $K.\frac{dh}{dr}$, and the discharge is given by

$$Q = \text{Area} \times \text{Velocity} = 2\pi\, rh.K.\frac{dh}{dr}$$

or $$h.dh = \frac{Q}{2\pi K} \cdot \frac{dr}{r}$$

Integration of the above equation with the known boundary c at the two observation bores yields.

$$\int_{h_1}^{h_2} h.dh = \frac{Q}{2\pi K} \int_{r_1}^{r_2} \frac{dr}{r}$$

$$\frac{(h_1^2 - h_2^2)}{2} = \frac{Q}{2\pi K} \cdot \ln \frac{(r_1)}{(r_2)}$$

or $$Q = \frac{\pi K(h_1^2 - h_2^2)}{\ln\left(\frac{r_1}{r_2}\right)}$$

where h_1 and h_2 are the water levels in the two observation w at the radial distances r_1 and r_2 respectively from the main well at which the cone of depression just commences is known as *of influence* denoted by R. If the radius of influence is known, th can be computed, even in the absense of observation wells using t condition at the well. That is

$$Q = \frac{\pi K(H^2 - h_w^2)}{\ln\left(\frac{R}{r_w}\right)}$$

where h_w and r_w are the water level in the well and radius respectively. But $K\,(H^2 - h_w^2) = \frac{2K\,(H + h_w)}{2} \cdot (H - h_w)$. And nothing but the drawdown in the well whereas $\frac{(H + h_w)}{2}$ m as the average thickness of the aquifer. Therefore $K\,(H^2 - h_w$ where T is the average transmissivity of the aquifer and s_w is t in the well. Now the discharge can also be expressed as

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$$Q = \frac{2\pi T s_w}{\ln\left(\frac{R}{r_w}\right)} \quad \text{...(9.15)}$$

The following empirical equation may sometimes be useful to predict the radius of influence R.

$$R = 3000\, s_w \sqrt{K} \quad \text{... (9.16)}$$

where s_w is in m and K is in m/s.

Equation (9.13) or (9.14) can be used (for a given drawdown) to estimate the yield from a well when the hydraulic conductivity is known, or to estimate the hydraulic conductivity of the aquifer if the discharge from the well is known.

This equation was derived by Dupuit in 1863 and modified by Thiem in 1906. It fails to describe accurately the drawdown curve near the well because the streamlines there are markedly curved, with large vertical flow components contradicting the assumption. Another serious drawback of this equation is that the true steady state equilibrium condition required for its applicability occurs only after a very long pumping period because of the low velocities of groundwater flow.

Example 9.5. *An unconfined aquifer has a thickness of 30 m. A fully penetrating 20 cm diameter well in this aquifer is pumped at a rate of 35 lit/s. The drawdown measured in two observation wells located at distances of 10 m and 100 m from the well are 7.5 m and 0.5 m respectively. Determine the average hydraulic conductivity of the aquifer. At what distance from the well the drawdown is insignificant ?*

Solution. $Q = \dfrac{\pi K(h_1^2 - h_2^2)}{\ln\left(\dfrac{r_1}{r_2}\right)}$

or $$K = \frac{Q \cdot \ln\left(\frac{r_1}{r_2}\right)}{\pi(h_1^2 - h_2^2)}$$

Here $Q = 35$ lit/s $= 0.035$ m^3/s

$r_1 = 100$ m ; $r_2 = 10$ m ; $\ln\left(\dfrac{r_1}{r_2}\right) = 2.302585$

$H = 30$ m ; $s_1 = 0.5$ m and $s_2 = 7.5$ m

$h_1 = H - s_1 = 30 - 0.5 = 29.5$ m

$h_2 = H - s_2 = 30 - 7.5 = 22.5$ m

$$\therefore \quad K = \frac{0.035 \times 2.302585}{\pi(29.5 + 22.5)(29.5 - 22.5)}$$

$= 7.04745 \times 10^{-5}$ m/s

$= 6.089$ m/day

Let R be the radius of influence where the drawdown is zero.

Then $$Q = \frac{\pi K(H^2 - h_1^2)}{\ln\left(\frac{R}{r_1}\right)}$$

$$\ln\left(\frac{R}{r_1}\right) = \frac{\pi K(H^2 - h_1^2)}{Q}$$

$$= \frac{\pi \times 7.04745 \times 10^{-5}(30^2 - 29.5^2)}{0.035}$$

$= 0.1882$

$\dfrac{R}{r_1} = 1.207$

$$\therefore \quad R = 1.207 \times 100 = 120.7 \text{ m.}$$

Example 9.6. *A well with a radius of 0.5 m, completely penetrates an unconfined aquifer of thickness 50 m and $K = 30$ m/day. The well is pumped so that the water level in the well remains at 40 m above the bottom. Assuming that pumping has essentially no effect on water table at $r = 500$ m, what is the steady-state discharge.*

Solution. $Q = \dfrac{\pi K(H^2 - h_w^2)}{\ln\left(\dfrac{R}{r_w}\right)}$

Here $H = 50$ m ; $h_w = 40$ m ; $R = 500$ m ; $r_w = 0.50$ m ; $K = 30$ m/day.

$$\therefore \quad Q = \frac{\pi \times 30 \times (50^2 - 40^2)}{\ln\left(\frac{500}{0.5}\right)} = 12279.387 \text{ m}^3\text{/day}$$

$= 0.1421225$ m^3/s

$= 142.12$ lit/s.

Steady Radial Flow to a Well in a Confined Aquifer. The flow around a well penetrating fully into a confined aquifer of thickness 'b' under steady-state conditions is shown in Fig. 9.8. In this case the depth of flow

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remains constant everywhere. The discharge flowing into the well through a section which is situated at a radial distane r from the well becomes

$$Q = 2\pi r b K \cdot \frac{dh}{dr}$$

$$dh = \frac{Q}{2\pi b K} \cdot \frac{dr}{r}.$$

Integration the above equation for the known boundary conditions at the two observation wells.

$$h_1 - h_2 = \frac{Q}{2\pi k b} \ln\left(\frac{r_1}{r_2}\right)$$

or
$$Q = \frac{2\pi T (h_1 - h_2)}{\ln\left(\frac{r_1}{r_2}\right)} \quad ...(9.17)$$

where $T = Kb$ is the transmissivity of the aquifer.

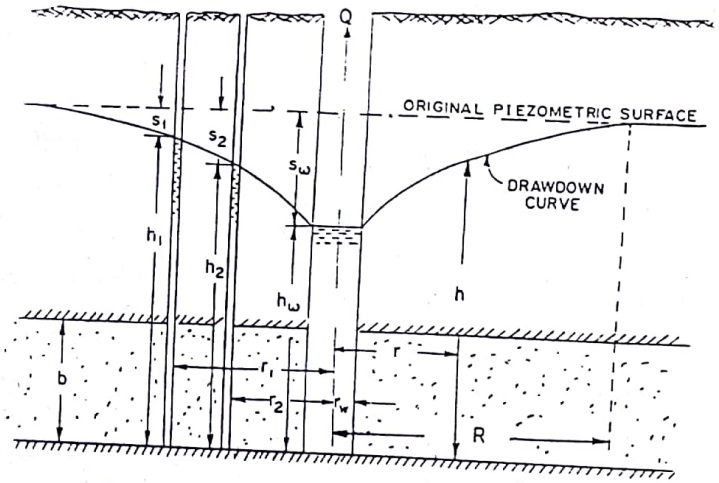

Fig. 9.8. Steady flow to a well in a confined aquifer.

In terms of drawdowns in the wells

$$Q = \frac{2\pi T (s_2 - s_1)}{\ln\left(\frac{r_1}{r_2}\right)} \quad ...(9.18)$$

In the absence of observation wells, utilising the boundary conditions at the well,

$$Q = \frac{2\pi T (H - h_w)}{\ln\left(\frac{R}{r_w}\right)} \quad ... (9.19)$$

where h_w and r_w are the height of water in the well and radius of the well, respectively and R is the radius of influence. Since $(H - h_w)$ represents the drawdown in the well s_w, the discharge may also be written as

$$Q = \frac{2\pi T s_w}{\ln\left(\frac{R}{r_w}\right)} \quad ... (9.20)$$

Example 9.7. *In an artesian aquifer of 8 m thick, a 10 cm diameter well is pumped at a constant rate of 100 lit/minute. The steady state drawdown observed in two wells located at 10 m and 50 m distances from the centre of the well are 3 m and 0.05 m respectively, compute the transmissivity and the hydraulic conductivity of the aquifer.*

Solution. $Q = \dfrac{2\pi T (s_2 - s_1)}{\ln\left(\dfrac{r_1}{r_2}\right)}$

$Q = 100$ lit/min $= 0.00167$ m^3/s

$r_2 = 10$ m ; and $r_1 = 50$ m

$\therefore \quad \ln\left(\dfrac{r_1}{r_2}\right) = 1.60944$

$(s_2 - s_1) = 3 - 0.05 = 2.95$ m

$$\therefore \quad T = \frac{0.00167 \times 1.60944}{2\pi \times 2.95} = 1.4472 \times 10^{-4} \text{ m}^2/\text{s}$$

$= 12.5$ m^2/day

The permeability of the aquifer

$$K = \frac{T}{b} = \frac{1.4472 \times 10^{-4}}{8}$$

$= 1.809 \times 10^{-5}$ m/s

$= 1.563$ m/day.

Example 9.8. *A well of 0.5 m diameter penetrates fully into a confined aquifer of thickness 20 m and hydraulic conductivity 8.2×10^{-4} m/s. What is the maximum yield expected from this well if the drawdown in the well is not to exceed 3 m. The radius of influence may be taken as 260 m.*

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Solution. $Q = \dfrac{2\pi T s_w}{\ln\left(\dfrac{R}{r_w}\right)}$

$T = K.b$

$= 8.2 \times 10^{-4} \times 20$

$= 0.0164 \text{ m}^2/\text{s}$

$s_w = 3 \text{ m}$;

$\dfrac{R}{r_w} = \dfrac{260}{0.25}$

$= 520$

$\ln\left(\dfrac{R}{r_w}\right) = 6.25383$

$\therefore \quad Q = \dfrac{2\pi \times 0.0164 \times 3}{6.25383}$

$= 0.04943 \text{ m}^3/\text{s}$

$= 49.43 \text{ lit/s}.$

9.6 UNSTEADY RADIAL FLOW TO A WELL IN A CONFINED AQUIFER

When pumping is started with an initially horizontal piezometric surface, the effect of discharge is to create a cone of depression in the piezometric surface near the well which extends outward with the continued pumping. In other words the position of the piezometric surface near the well changes continuously with time. This is clearly a case of unsteady flow. Theis, in 1935, developed the non-steady-state analysis of flow in a confined aquifer. Theis equation is derived from the analogy between the flow of ground water and conduction of heat and it is based on the following assumptions.

(*i*) the aquifer is confined,

(*ii*) the well diameter is very small so that the storage within the well can be neglected, and

(*iii*) the aquifer is dewatered instantaneously as the piezometric head drops. The equation is given as

$$s = \frac{Q}{4\pi T} \int_u^\infty \frac{e^{-u}}{u} . du \qquad \text{... (9.21)}$$

where s = The drawdown in m in an observation well at a distance r from the pumping well after 't' seconds since the pumping began.

Q = The constant pumping rate from the well in m^3/s

T = The transmissivity of the aquifer in m^2/s

and $u = \dfrac{r^2 S}{4Tt}$ ... (9.22)

where S is the storage coefficient of the aquifer. If a time unit of day is preferred t, T and Q should be expressed in days, m^2/day and m^3/day, respectively.

The exponential integral in Eq. (9.21) is symbolically written as $W(u)$ and is generally called *well function* or *Theis well function*. In terms of well function Eq. (9.21) becomes

$$s = \frac{Q}{4\pi T} W(u) \qquad \text{... (9.23)}$$

The well function can be evaluated from the series expansion given by

$$W(u) = \left[-0.577216 - \ln u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!} + \ldots\ldots\right] \text{... (9.24)}$$

The well function is tabulated in Table 9.2. For example, for $u = 0.005$ the value of $W(u)$ may be read as 4.73 from this table. In the Theis equation it is assumed that the confining beds are water tight and do not transmit any water. But in the case of leaky aquifers there may be considerable flow through the confining beds. Modified equations have been developed which take into account such leakage. Equations are also available for unsteady radial flow to partially penetrating wells in unconfined aquifers. However, they are not discussed here.

The Theis equation can be used to evaluate the average aquifer parameters S and T in the viscinity of the well by measuring the changes in the drawdown with time in one or more observation wells near the pumping well under the influence of constant pumping rate. Pumping need not be continued for long duration till the so-called steady state conditions are attained.

Because of the mathematical difficulties encountered in applying Eq. (9.21) or its equivalent Eq. (9.24), several investigators have developed simple approximate solutions that can be readily applied for field puposes. Three methods by Theis, Cooper and Jacob, and Chow and the Theis recovery test method are only discussed here.

Theis Method. Eq. (9.22) can be rearranged to take the form

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Table 9.2. Well Function : Values of W(u) for Values of u

u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
$\times 1$	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
$\times 10^{-1}$	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
$\times 10^{-2}$	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
$\times 10^{-3}$	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
$\times 10^{-4}$	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
$\times 10^{-5}$	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
$\times 10^{-6}$	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
$\times 10^{-7}$	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
$\times 10^{-8}$	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
$\times 10^{-9}$	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
$\times 10^{-10}$	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
$\times 10^{-11}$	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
$\times 10^{-12}$	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
$\times 10^{-13}$	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
$\times 10^{-14}$	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
$\times 10^{-15}$	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

$$\frac{r^2}{t} = \left(\frac{4T}{S}\right) u \qquad \text{... (9.25)}$$

Similarly Eq. (9.23) can be written as

$$s = \left(\frac{Q}{4\pi T}\right) W(u) \qquad \text{[from Eq. (9.23)]}$$

Comparing these two equations it can be seen that the relation between $W(u)$ and u must be similar to that between s and $\frac{r^2}{t}$, because the terms in the parenthesis in both the equations are constants. Based on this similarity Theis suggested a graphical procedure to obtain an approximate solution for S and T.

The procedure is as follows. During a pumping test the drawdown in an observation well is continuously measured with time. A curve is plotted on a transparent logarithmic paper between drawdown s and the values of $\frac{r^2}{t}$. This curve is known as the *data curve*. Another curve is prepared on a logarithmic paper between the values of u and $W(u)$ obtained from Table 9.2, which is called the *type curve*. Of course the scale for one log cycle adopted for both these curves must be same. The data curve is now superimposed on the type curve such that the axes of both curves are parallel and it is then adjusted until some portion of the data curve coincides with the type curve. This procedure is called the *matching*. The cordinates of any convenient point on the matched portion namely u, $W(u)$, $\frac{r^2}{t}$ and s are determined. Then the values of T and S can be obtained by substituting these coordinate values in Eqs. (9.23) and (9.25).

Example 9.9. *A 20 cm diameter well penetrating a confined aquifer is pumped at a uniform rate of 30 lit/s and the drawdowns measured in an observation well 60 m away are given below.*

t, hours	0.0167	0.0333	0.0666	0.1333	0.5	1.0	2.0	3.0	4.0
s, metres	0.20	0.30	0.41	0.53	0.76	0.90	1.00	1.07	1.12

Determine the aquifer parameters T and S using the Theis method and estimate the drawdown in the well at the end of 7 days of pumping.

Solution. Values of r^2/t in m^2/min are computed using $r = 60$ m and they are shown in Table 9.3.

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Table 9.3. Pumping Test Data of Example 9.9 (r = 60 m)

t in hours	s in m	r^2/t in m^2/min
0.0167	0.20	3593
0.0333	0.30	1802
0.0666	0.41	901
0.1333	0.53	450
0.5	0.76	120
1.0	0.90	60
2.0	1.00	30
3.0	1.07	20
4.0	1.12	15

Values of s and r^2/t are plotted on a logarithmic paper. Values of $W(u)$ and u from Table 9.2 are plotted on another sheet of logarithmic paper and a curve is drawn through the points. The two curves are superposed

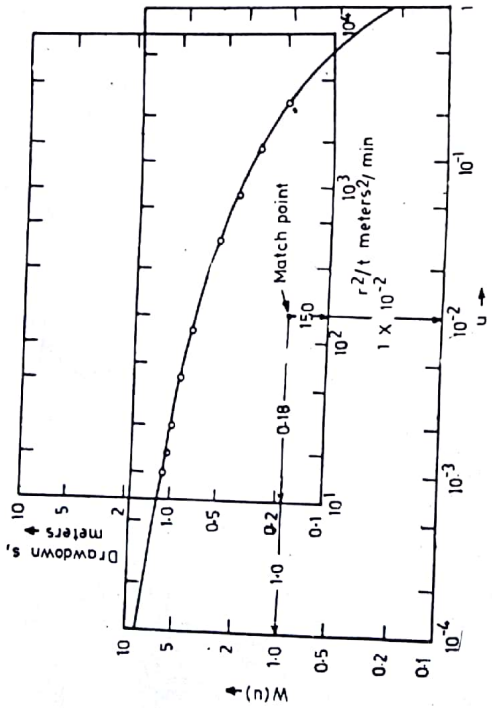

Fig 9.9. Theis method of solution for S and T.

and shifted with coordinate axes parallel until the curve of the observational points (data curve) coincides with the type curve, as shown in Fig 9.9. For convenience a match point is selected with $W(u) = 1.00$ and $u = 1 \times 10^{-2}$ for which s and r^2/t are read as 0.18 m and 150 m^2/min respectively. Now from Eq. (9.23)

$$T = \frac{Q}{4\pi s} . W(u)$$

$$Q = 30 \text{ l/s} = 0.03 \text{ m}^3/\text{s} = 2592 \text{ m}^3/\text{day}$$

$$\therefore \quad T = \frac{2592 \times 1}{4\pi \times 0.18} = 1145.92 \text{ m}^2/\text{day}.$$

and from Eq. (9.25),

$$S = \frac{4Tu}{(r^2/t)}$$

$$r^2/t = 150 \text{ m}^2/\text{min} = 216000 \text{ m}^2/\text{day}$$

$$\therefore \quad S = \frac{4 \times 1145.92 \times 1 \times 10^{-2}}{216000} = 0.0002122.$$

From Eq. (9.25),

$$u = \frac{S}{4T} \cdot \frac{r^2}{t}$$

At the end of seven days, $\frac{r^2}{t} = \frac{60 \times 60}{7} = 514.286 \text{ m}^2/\text{day}$

and $u = \frac{0.0002122}{4 \times 1145.92} \times 514.286 = 2.3809 \times 10^{-5}$

From Table 9.2, when $u = 2.3809 \times 10^{-5} W(u) = 10.088$

$$s = \frac{Q}{4\pi T} W(u) = \frac{2592}{4\pi \times 1145.92} \times 10.088 = 1.816 \text{ m}.$$

Cooper and Jacob Method. It was noted by Cooper and Jacob that for small values of r and large values of t, u is small, so that the series terms in Eq. (9.24) become negligible after the first two terms. Let s_1 and s_2 be the drawdowns in an observation well near the pumping well at two different times t_1 and t_2 where both of them are large. Then, we have

$$s_1 = \frac{Q}{4\pi T} W(u_1)$$

where $u_1 = \frac{r^2 S}{4Tt_1}$

and $$s_2 = \frac{Q}{4\pi T} W(u_2)$$

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where $u_2 = \dfrac{r^2 S}{4Tt_2}$

Therefore, $\Delta s = s_2 - s_1$

$$= \frac{Q}{4\pi T}[W(u_2) - W(u_1)]$$

Since u_1 and u_2 are small, from the approximation mentioned above

$$\Delta s = \frac{Q}{4\pi T}[-0.5772 - \ln u_2 - (-0.5772 - \ln u_1)]$$

$$= \frac{Q}{4\pi T}(\ln u_1 - \ln u_2)$$

$$= \frac{Q}{4\pi T} \ln\left(\frac{u_1}{u_2}\right)$$

$$= \frac{Q}{4\pi T} \ln\left(\frac{t_2}{t_1}\right)$$

or $$T = \frac{Q}{4\pi \Delta s} \ln\left(\frac{t_2}{t_1}\right) \quad \text{... (9.26)}$$

Changing this to decimal logarithm

$$T = \frac{2.3026\ Q}{4\pi \Delta s} \log\left(\frac{t_2}{t_1}\right) \quad \text{... (9.27)}$$

The drawdown s is plotted on an arithmetic scale against time t on a logarithmic scale as shown in Fig. 9.10. This time-drawdown curve will tend to be a straight line for large values of t (or small values of u). If Δs is taken as the drawdown difference for one log cycle on time, that is $t_2 = 10\ t_1$, T is easily evaluated from

$$T = \frac{2.3026\ Q}{4\pi \Delta s} \quad \text{...(9.28)}$$

since $\log\left(\dfrac{t_2}{t_1}\right) = 1.$

To evaluate S, the following procedure is adopted. Assuming the present approximation holds good for the entire range,

$$s = \frac{Q}{4\pi T} W(u)$$

When s is zero, $W(u)$ must be equal to zero. That is

$$0 = W(u)$$

$$= -0.5772 - \ln u.$$

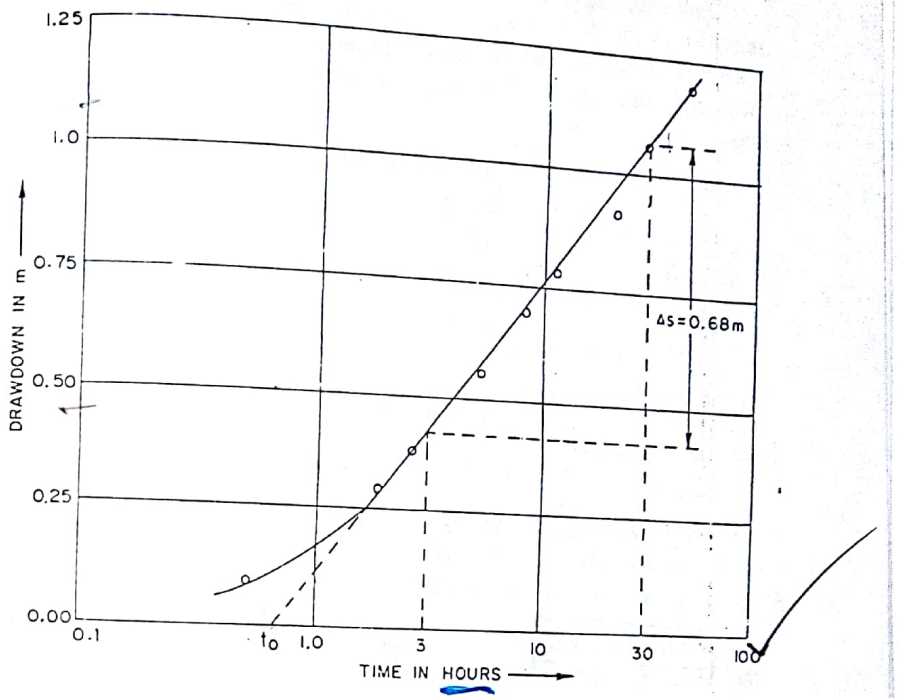

Fig. 9.10. Time-drawdown plot for Cooper-Jacob method.

which yields $u = 0.56146$. The straight line portion of the curve is extended back to intersect the x-axis to give t_0, the time corresponding to $s = 0$. Hence at $s = 0$,

$$u = 0.56146 = \frac{Sr^2}{4Tt_0}$$

or $$S = \frac{2.246\ Tt_0}{r^2} \quad \text{...(9.29)}$$

Equations (9.28) and (9.29) are used to evaluate T and S after obtaining Δs and t_0 from the time-drawdown curve.

Example 9.10. *A pumping test was carried out on a new irrigation bore well penetrating fully into a confined aquifer at a rate of 22 lit/s. The drawdown measured in an observation well located at 45.7 m from the pumping well during the test is as given below. Determine T and S of the aquifer, using Cooper-Jacob method.*

Time t in hours	0.5	1.8	2.7	5.4	9.0	12.0	18	30	54
Drawdown s in m	0.091	0.294	0.382	0.55	0.701	0.785	0.911	1.06	1.24

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Solution. The given time-drawdown data is plotted on a semi-log graph as shown in Fig. 9.10. From this graph Δs corresponding to t_1 and t_2 of 3 hrs and 30 hrs is read as 0.68 m. From the same graph by extending the straight line portion of the curve to intersect x-axis, t_0 is read as 0.8 hours.

$$Q = 22 \text{ lit/s} = 0.022 \text{ m}^3/\text{s}$$

$$T = \frac{2.3026\,Q}{4\pi\Delta s} = \frac{2.3026 \times 0.022}{4\pi \times 0.68}$$

$$= 5.928 \times 10^{-3} \text{ m}^2/\text{s} = 512.2 \text{ m}^2/\text{day}$$

$$S = \frac{2.246\,Tt_0}{r^2}$$

$$t_0 = 0.8 \text{ h} = 2880 \text{ seconds}$$

$$r = 45.7 \text{ m}$$

$$\therefore \quad S = \frac{2.246 \times 5.928 \times 10^{-3} \times 2880}{45.7 \times 45.7}$$

$$= 0.01836$$

Chow's Method. Ven Te Chow developed a method of solution with the advantage of avoiding curve fitting and being unrestricted in its application. According to this method, the data on time-drawdown in an observation well is plotted on a semilogarithic paper in the same manner as for Cooper-Jacob method. On the plotted curve an arbitrary point with co-ordinates t and s is chosen. At this chosen point a tangent is drawn to the drawdown curve and the drawdown difference Δs in m per log cycle is determined. Then the value of the new function $F(u)$ is computed from

$$F(u) = \frac{s}{\Delta s} \qquad \ldots (9.30)$$

And the values of u, and $W(u)$ corresponding to $F(u)$ are found from Fig. (9.11) given by Chow.

Since the relation between $F(u)$, $W(u)$ and u is given by

$$F(u) = \frac{W(u)\,e^u}{2.3} \qquad \ldots (9.31)$$

the value of u may also be found out by trial and error without resorting to Fig.9.9 such that along with the corresponding value of $W(u)$ obtained from well function table, it satisfies Eq. (9.31). If $F(u) > 2.0$; even the trial and error procedure is avoided since in that case $W(u) = 2.3\ F(u)$ and u is then obtained from well function table.

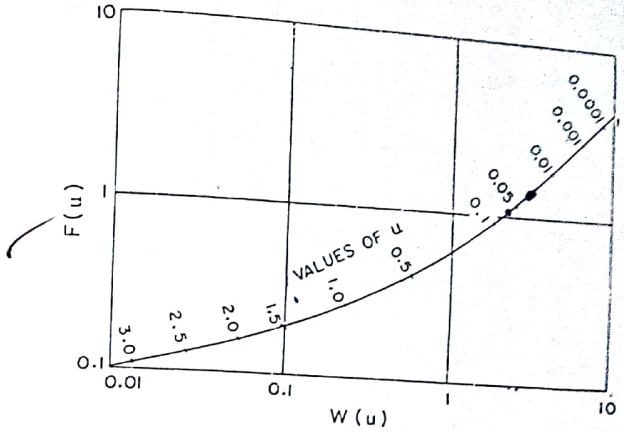

Fig. 9.11 Relation between $F(u)$, $W(u)$ and u for Chow's method.

After knowing u and $W(u)$, the aquifer parameters T and S are finally computed using Eqs. (9.23) and (9.25).

Example 9.11. *The time-drawdown measurements in an observation well at a radial distance of 200 m from the pumping well are as given under. What are the values of the parameters T and S if the constant rate of pumping is 1000 m³/day? Use Chow's method.*

t in days	0.001	0.005	0.01	0.05	0.10	0.50	1.0	5.0	10
s in m	0.017	0.097	0.145	0.267	0.322	0.449	0.504	0.632	0.687

Solution. The given s *vs* t data is plotted on a semilogarithmic paper as shown in Fig. 9.12. At point A, with co-ordinates of $s = 0.2$ m and $t = 0.02$ days, a tangent is drawn which gives $\Delta s = 0.18$ m for one log cycle.

$$\therefore \quad F(u) = \frac{s}{\Delta s}$$

$$= \frac{0.2}{0.18} = 1.11$$

From Fig. (9.11), when $F(u) = 1.11$

$$W(u) = 2.2 \text{ and } u = 0.065$$

From Eq. (9.23), $$T = \frac{Q}{4\pi s}\,W(u)$$

$$= \frac{1000 \times 2.2}{4\pi \times 0.2}$$

$$= 875.35 \text{ m}^2/\text{day}$$

$$= 0.01 \text{ m}^2/\text{s}$$

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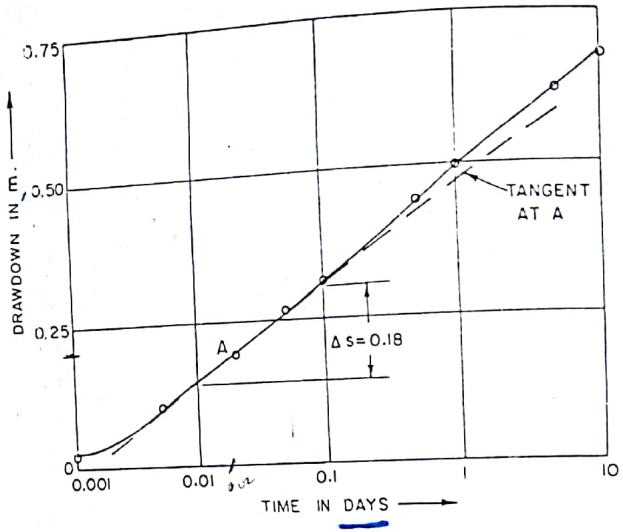

Fig. 9.12. Determination of T and S by Chow's method.

From Eq. (9.25), $$S = \frac{4\,Tu.t}{r^2}$$

$$= \frac{4 \times 875.35 \times 0.065 \times 0.02}{200 \times 200}$$

$$= 1.14 \times 10^{-4}$$

Recovery test. At the end of a pumping test, when the pump is stopped, the water levels in pumping and observation wells will begin to rise. This is referred to as the *recovery* of ground-water. The measurements of drawdown below the original static water level (prior to pumping) during the recovery period are known as *residual drawdowns*, denoted by s'. Theis showed that for r small and t' large the residual drawdown s' can be given as,

$$s' = \frac{2.303\ Q}{4\pi T} \log\left(\frac{t}{t'}\right) \qquad \text{... (9.32)}$$

where t is the time since pumping was started and t' is the time since pumping was stopped. That is, t is the total time of pumping plus t'. Thus, a plot of residual drawdown s' versus the logarithm of (t/t') plots as a straight line, the slope of which equals $2.303\ Q/4\pi T$. So, if $\Delta\ s'$ is the residual drawdown per log cycle of t/t', then the transmissivity T can be directly evaluated as

$$T = \frac{2.303\ Q}{4\pi \Delta s'} \qquad \text{... (9.33)}$$

The transmissivity obtained from recovery test can be used as a check on T calculated from drawdown data during pumping. However, the recovery test cannot determine the value of S.

9.7 UNSTEADY FLOW TO A WELL IN UNCONFINED AQUIFER

Unsteady flow into a well in an unconfined aquifer is more complicated since the transmissivity of the aquifer changes with time t and radial distance r. Also vertical flow components may be significant near the well invalidating Dupuit-Forchheimer assumptions. However, if the drawdown s is small compared to the thickness of the aquifer H, the Theis Eq. (9.23) of confined aquifers can be used for unconfined aquifers. For larger drawdowns more exact solutions are available.

9.8 EFFECTS OF BOUNDARIES ON GROUNDWATER MOVEMENT

In the analysis of steady and unsteady flows towards a well in an aquifer, it has been assumed that the aquifer is of infinite extent having uniform thickness, and the water table or piezometric surface is horizontal before the pumping commenced. Such ideal conditions are rearly met with in the field although in many cases the conditions are fairly satisfactory to yield results of reasonable accuracy. In several other cases the boundary conditions are far from ideal and need some special attention. For example, when two wells are closely located and simultaneously pumped the effect of pumping in each will be felt by other and the cone of depression will not be symmetrical. The water table in the case of unconfined aquifers may have significant slope because of various topographic conditions. A recharge source such as a stream may exist within the range of well drawdown limits.

The *method of images* is a convenient way to treat many of the boundary problems encountered in the field. A detailed description of the method of images can be found in any text book exclusively dealing with groundwater hydrology.

9.9 GROUNDWATER EXPLORATION

The occurrence and movement of groundwater are closely related to the geological formations. Therefore a thorough understanding of geology of the area under investigation will be of immense help to the hydrologists in search of groundwater. Apart from drilling, which is the direct method, there are some geophysical methods which can be used to detect the presence of groundwater. These include electrical resistivity method and seismic refraction method.

In the electrical resistivity method, the resistivity of the formation is measured by sending electricity through it from the ground surface. In the seismic refraction method a seismic shock is produced by striking the ground

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Solution

Ground water storage available annually

$$Q = \text{Area} \times \text{Depth of fluctuation of g.w.t.} \times \text{Specific yield}$$

$$= 10^6 \times 3.2 \times \frac{2}{100} = 64{,}000 \text{ m}^3$$

which can be replenished by normal rainfall whose volume, assuming an infiltration rate of $10\% = 10^6 \times \frac{700}{1000} \times \frac{10}{100} = 70{,}000$ m³ and also as observed by the normal fluctuation of water table. Assuming a per capita consumption of 180 lpd annual drinking water supply required $= 154 \times 180 \times 365 = 10{,}120{,}000$ litres or 10,120 m³.

The annual drinking water supply required is 10,120 m³ against an availability of 64,000 m³. Thus there is enough replenishable ground water resource available in the area to meet the drinking water needs of the local population. The only problem is location and economic construction of potential wells.

Example 4.3: In a phreatic aquifer extending over 1 km² the water table was initially at 25 m below ground level. Sometime after irrigation with a depth of 20 cm of water, the water table rose to a depth of 24 m b.g.l. Later 3×10^5 m³ of water was pumped out and the water table dropped to 26.2 m b.g.l. Determine (i) specific yield of the aquifer, (ii) deficit in soil moisture (below field capacity) before irrigation.

Solution

Volume of water pumped out = Area of aquifer × drop in g.w.t. × specific yield

$$3 \times 10^5 = 10^6 \times 2.2 \times S_y$$

$$S_y = 0.136, \text{ or } 13.6\%$$

Volume of irrigation water recharging the aquifer = Area of aquifer × rise in g.w.t × S_y

Considering an area of 1 m² of aquifer,

$$1 \times y = 1 \times 1 \times 0.136$$

Recharge volume (depth) $y = 0.136$ m, or 136 mm

Soil moisture deficit (below field capacity) before irrigation $= 200 - 136 = 64$ mm

Example 4.4: In an area of 100 ha, the water table dropped by 4.5 m. If the porosity is 30% and the specific retention is 10% determine (i) the specific yield of the aquifer, (ii) change in ground water storage.

Solution

$$\text{Porosity} = S_y + S_r$$

$$30\% = S_y + 10\%$$

$$S_y = 30 - 10$$

$$= 20\% \text{ or } 0.2$$

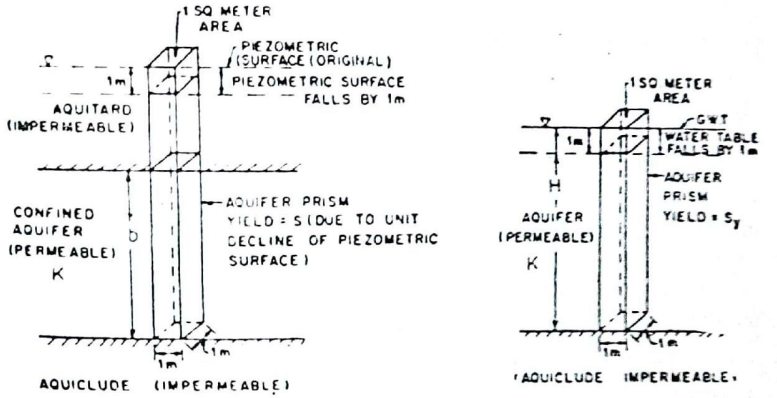
Change in ground water storage = Area of aquifer × drop in g.w.t. × S_y

$$= 100 \times 4.5 \times 0.2$$

$$= 90 \text{ ha-m, or } 90 \times 10^4 \text{ m}^2$$

Storage Coefficient

Storage coefficient of an aquifer is the volume of water discharged from a unit prism, i.e., a vertical column of aquifer standing on a unit area (1 m²) as water level (piezometric level in confined aquifer—artesian conditions) falls by a unit depth (1 m). For unconfined aquifers (water table conditions) the storage coefficient is the same as specific yield, Fig. 4.4. The storage coefficient for confined aquifers ranges from 0.00005 to 0.005 and for water table aquifers 0.05 to 0.30.



a. Storage coefficient S of a confined or artesian aquifer

b. Storage coefficient or specific yield (S_y) of a water table or unconfined aquifer

Fig. 4.4 Diagrammatic representation of coefficient of storage.

Under artesian conditions, when the piezometric surface is lowered by pumping, water is released from storage by the compression of the water bearing material (aquifer) and by expansion of the water itself*. Thus the

*As the water comes out of the pores into the well under atmospheric pressure.

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coefficient of storage is a function of the elasticity of water and the aquifer skeleton and is given by (Jacob, 1950) as

$$S = \gamma_w b(\alpha + n\beta) \qquad (4.4)$$

where S = coefficient of storage, fraction; n = porosity of aquifer, fraction; b = saturated thickness of aquifer (m); γ_w = units weight of water (9810 N/m³); $\beta = \frac{1}{K_w}$, reciprocal of the bulk modulus of elasticity of water $K_w = 2.1$ GN/m² $= 2.1 \times 10^9$ N/m²; and $\alpha = \frac{1}{E_s}$, reciprocal of the bulk modulus of elasticity of aquifer skeleton.

The fraction of storage attributable to expansibility water

$$S_w = 4.7 \times 10^{-6}\, nb \qquad (4.5)$$

The bulk modulus of compression of some formation material are given in Table 4.3.

Table 4.3 Bulk modulus of compression of formation materials

Material	Bulk modulus of compression E_s, N/m²
Plastic clay	$5\text{-}40 \times 10^5$
Stiff clay	40-80
Medium-hard clay	80-150
Loose sand	100-200
Dense sand	500-800
Dense sandy gravel	1,000-2,000
Rock—fissured, jointed	1,500-30,000
Rock—sound	>30,000

Example 4.5: An artesian aquifer 20 m thick has a porosity of 20% and bulk modulus of compression 10^8 N/m². Estimate the storage coefficient of the aquifer. What fraction of this is attributable to the expansibility of water.

Solution

$$S = \gamma_w b\,(\alpha + n\beta)$$

$$= 9810 \times 20\left(\frac{1}{10^8} + 0.20 \times \frac{1}{2.1 \times 10^9}\right)$$

$$= (1.962 + 0.0187)\ 10^{-3}$$

$$= 1.98 \times 10^{-3},$$

say, $\mathbf{2 \times 10^{-3}}$

The fraction of storage attributable to the expansibility of water (taking only the second term within the brackets)

$$S_w = 0.0187 \times 10^{-3}$$

$$= \frac{1.87 \times 10^{-5}}{1.98 \times 10^{-3}} \text{ of } S$$

$$\approx \frac{1}{100} \text{ of } S, \quad \text{or } \mathbf{1\% \text{ of } S}$$

Example 4.6: In a certain place in Andhra Pradesh, the average thickness of the confined aquifer is 30 m and extends over an area of 800 km². The piezometric surface fluctuates annually from 19 m to 9 m above the top of the aquifer. Assuming a storage coefficient of 0.0008, what ground water storage can be expected annually?

Assuming an average well yield of 30 m³/hr and about 200 days of pumping in a year, how many wells can be drilled in the area?

Solution:

$$\Delta GWS = A_{aq} \times \Delta\text{piezo. Surface} \times S = (800 \times 10^6)(19 - 9)\ 0.0008$$

or $\quad 6.4 \times 10^6$ m³, **or 6.4 M m³**

$$\text{Annual draft} = (30 \times 24)\ 200 = 0.144 \times 10^6 \text{ m}^3$$

or $\quad 0.114$ M m³

$$\text{Number of wells that can be drilled in the area} = \frac{6.4}{0.144}$$

$$= 44.5, \text{ say } \mathbf{44 \text{ wells}}$$

Of course, the well sites have to be investigated and there should be sufficient spacing for the wells.

Example 4.7: An aquifer has an average thickness of 60 m and an aerial extent of 100 ha. Estimate the available ground water storage if

(a) the aquifer is unconfined and the fluctuation in *GWT* is observed as 15 m,

(b) the aquifer is confined, and the piezometric head is lowered by 50 m, which drains half the thickness of the aquifer.

Assume a storage coefficient of 2×10^{-4} and a specific yield of 16%.

Solution: (a) $\Delta GWS = A_{aq}.\ \Delta GWT.\ S_y = 100$ ha $\times 15$ m (0.16)

$= \mathbf{240}$ **ha-m**

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(b) $\Delta GWS = A_{aq}[\underbrace{\Delta \text{ piezo. head} \times S}_{\text{(as confined)}} + \underbrace{\Delta GWT \times S_y}_{\text{(as unconfined)}}]$

$= 100 \text{ ha } [20\,(2 \times 10^{-4}) + 30(0.16)]$

$= \mathbf{480.4 \text{ ha-m}}$

Land Subsidence Due to Ground Water Withdrawals

In areas where unconsolidated and semi-consolidated alluvial aquifers are confined or partially confined by thick fine-grained beds, subsidence of the land surface is likely to result as artesian pressure is reduced by ground water pumpage. Subsidence can be detected by careful relevelling of surface bench marks. Compaction of sediments may extend to depths more than 300 m and is directly related to declines in artesian pressure.

Reduction of artesian pressure results in land subsidence that may cause failure of wells, settlement of buildings. In Southern California and in larger areas near Mexico city, subsidence has exceeded 3 m.

The elastic subsidence of the land surface can be computed from the equation (Lohman, 1961)

$$\Delta b = \Delta p \left(\frac{S}{\gamma_w} - nb\beta \right) \tag{4.6}$$

where Δb = land subsidence (m); Δp = reduction in artesian pressure (N/m^2) and γ_w = specific weight of water.

Example 4.8: Estimate the probable land subsidence when the piezometric head drops by 70 m in an artesian aquifer 30 m thick, having a porosity of 30% and storage coefficient 2×10^{-4}.

Solution

$$\Delta b = \Delta p \left(\frac{S}{\gamma_w} - nb\beta \right)$$

$$= 9810 \times 70 \left[\frac{2 \times 10^{-4}}{9810} - 0.30 \times 30 \times \frac{1}{2.1 \times 10^{9}} \right]$$

$= 0.011$ m, or **11 mm**

which may cause damage to canal linings under water table conditions. When the water table is lowered by pumping, ground water is obtained from storage by gravity drainage of the interstices in the portion of the aquifer unwatered by pumping.

Specific capacity: Specific capacity of a well is its discharge per unit drawdown usually expressed in lpm/m. This is a measure of the effectiveness of a well.

Safe yield: Safe yield from a well is the amount of water that can be economically withdrawn from the well in the foreseeable future without

causing depletion of the aquifer. Tapping of ground water (location, spacing and yield) in a well field should be so phased that the recharge and discharge of the aquifer are almost balanced without causing an overdraft in the area.

Aquifers: The water bearing geologic formations or strata which yield significant quantity of water for economic extraction from wells are called aquifers.

Rock formations that serve as good aquifers are:

Gravel, sand and sandstone, alluvium
Limestone with cavities formed by the action of ucid waters
Marble with fissures and cracks
Granite rocks with fissures and joints
Weathered gneisses and schists
Heavily shattered quartzites
Vesicular basalts
Slate (better than shale owing to its jointed conditions)

Aquiclude: A geologic formation which can only store water but cannot transmit significant amounts; e.g. clay lenses, shale, etc.

Aquifuge: A geologic formation with no interconnected pores and hence can neither absorb nor transmit water; e.g. basalt, granite, etc.

Aquitard: A geologic formation of a rather impervious nature which transmits water at a slow rate compared to an aquifer but insufficient to supply individual wells; e.g. clay lenses interbedded with sand.

Transmissibility coefficient: The coefficient of transmissibility (T) is the discharge through unit width of aquifer for the fully saturated depth under a unit hydraulic gradient and is usually expressed as lpd/m or m^2/sec. It is the product of field permeability (K) and saturated thickness of the aquifer (b); $T = Kb$ and has the dimensions L^2/T.

Permeability: Permeability (K) is the ability of a formation to transmit water through its pores when subjected to a difference in head. It can be defined as the flow per unit cross-sectional area of the formation when subjected to a unit hydraulic head per unit length of flow (i.e. per unit hydraulic gradient) and has the dimension of velocity, i.e. L/T.

Movement of Ground Water

Ground water moves from levels of higher energy to levels of lower energy, its energy being essentially the result of elevation and pressure, the velocity heads being neglected since the flow is essentially laminar. The velocity is very small in the laminar range—of the order of 1 cm/sec—and the Reynolds number (Re) for ground water flow (Hantush, 1964) varies from 1 to 10 and is given by

$$\text{Re} = \rho v \frac{d_m}{\mu} \tag{4.7}$$

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where v = velocity (seepage or bulk) of ground water flow; d_m = mean diameter of the soil grains (usually taken as D_{10}); ρ = density of ground water and μ = dynamic viscosity of ground water.

Velocity of ground water flow which is entirely laminar is given by Darcy's law which states that 'the velocity of flow in a porous medium is proportional to the hydraulic gradient', Fig. 4.5.

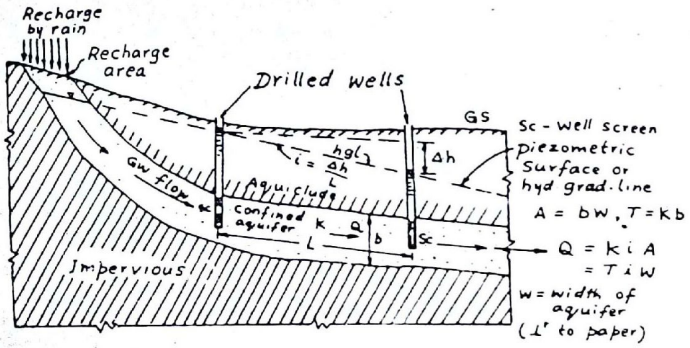

Fig. 4.5 Ground water flow.

$$v = Ki \tag{4.8}$$

where K = coefficient permeability and i = hydraulic gradient

$$\left(= \frac{h}{L} \text{ if a head } h \text{ is lost in a length } L\right).$$

Ground water flow $Q = Av$

$$= AKi, \qquad A = wb, \quad T = kb$$

Therefore

$$Q = Tiw \tag{4.9}$$

where A = cross-sectional area of the aquifer; w = width of the aquifer; b = saturated thickness of the aquifer and T = transmissibility of the aquifer, i.e. capacity of a unit prism of aquifer to transmit water.

The actual velocity (v_a) at which the water is moving through an aquifer, i.e. on an average, the velocity at which a tracer would move through a permeable medium, is given by

$$v_a = \frac{Q}{A_{act}} = \frac{Q}{nA} = \frac{v}{n} \tag{4.10}$$

where $v(= Q/A)$ is the apparent or seepage velocity given by Darcy's law $v = Ki$.

Flow in coarse-grained aquifer under high drawdown, especially in the area adjacent to the pumping well is given by the non-Darcy regime of flow generally described by the Forchheimer equation (Ahmed, 1969)

$$i = av + bv^2 \tag{4.10a}$$

where a and b are constants depending upon the properties of porous media and the fluid, and have the dimensions

$$[a] = \frac{1}{[v]} = \frac{T}{L}, \qquad [b] = \frac{1}{[v^2]} = \frac{T^2}{L^2},$$

where [ ] means 'dimensions of'. That is, the flow is no longer laminar, particularly as it arrives at the well face due to high gradients and exhibits non-linear relationship between the velocity and hydraulic gradient. While the use of Darcy's law is valid for low Reynolds number, typically $R_e < 1$, various flow situations have been observed where the Reynolds number of flow is likely to be greater than unity.* For example in a gravel packed well (mean size of gravel $\approx$ 5 mm), $R_e \approx 45$ and the flow would be transitional at a distance of about 5 to 10 times the well radius.

Example 4.9: It was observed in a field test that 3 hr 20 min was required for a tracer to travel from one well to another 20 m apart, and the difference in their water surface elevations was 0.5 m. Samples of the aquifer between the wells indicated a porosity of 15%. Determine the permeability of the aquifer, seepage velocity, and the Reynolds number for the flow, assuming an average grain size of 1 mm and ν_{water} at 27°C = 0.008 Stoke.

Solution.

Actual velocity of flow through the aquifer as indicated by the tracer

$$v_a = \frac{x}{t} = \frac{20\text{ m}}{3\frac{1}{3}\text{ hr}} = 6\text{ m/hr}$$

Seepage velocity as given by Darcy's law,

$$v = Ki = K\frac{h}{L} = K\frac{0.5\text{ m}}{20\text{ m}} = \frac{K}{40}$$

From Eq. 4.10, $v_a = v/n$

$$6 = \frac{K}{40\,(0.15)}$$

K = 36 m/hr at the field temperature

= 864 m/day or m³/day/m²

= 8.64×10^6 lpd/m²

*In aquifers containing large diameter solution openings, coarse gravels, and also in the immediate vicinity of a gravel packed well, flow is no longer laminar.

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Seepage velocity, $v = \frac{K}{40} = \frac{864}{40}$

$= 21.6$ m/day, or 0.025 cm/sec

Reynolds number,

$$\text{Re} = \frac{v d_m}{\nu} = \frac{0.025}{100} \times \frac{1}{1000} \times \frac{1}{0.008 \times 10^{-4}} = 0.3$$

Example 4.10: A 30 cm gravel packed well is pumped at a constant rate of 5000m³/hr from a confined aquifer of thickness 50 m, having $D_{10} = 0.23$ mm, $D_{60} = 0.6$ mm.

(i) What is the domain around the well for which Darcy's law is applicable, assuming that the law is valid up to $R_e = 6$? $\nu_{water} = 1$ c St

(ii) If the gravel pack is 14 cm thick and has $D_{10} = 1.5$ mm and $D_{60} = 3$ mm, estimate the Reynold's number and the seepage gradient at mid-pack.

Solution:

(i) $R_e = \frac{vD}{\nu}$; $D = D_{60}$ (mean size)

$$6 = \frac{v(0.6 \times 10^{-3})}{1 \times 10^{-6}} \qquad v = 0.01 \text{ m/s}$$

$Q = Av$

$$\frac{5000}{60 \times 60} = 2\pi\, r.50\,(0.01)$$

$r = 0.442$ m, say 44 cm

Hence Darcy's law is valid beyond $r = 44$ cm

(ii) At $r = \frac{30}{2} + \frac{14}{2} = 22$ cm (mid-gravel pack),

$$\frac{5000}{60 \times 60} = 2\pi\,(0.22)\,50\,v,\ v = 0.02 \text{ m/s}$$

$$R_e = \frac{vD}{\nu} = \frac{0.02\,(3 \times 10^{-3})}{1 \times 10^{-6}} = 60$$

Actually, $R_e > 60$, since the depth of flow near the well is < 50m, due to the cone of depression.

$$K = C\,D_{10}^2, \text{ Allen Hazen}$$

$$\frac{K_g}{K_a} = \left(\frac{1.5}{0.23}\right)^2 = 36$$

where K_g and K_a are the permeabilities of the gravel pack and aquifer, respectively, $C = 100$ where D_{10} is in cm, K in cm/s,

$$K_a = 100\,(0.023)^2 = 0.053 \text{ cm/s}$$

$$K_g = 0.053 \times 36 = 1.91 \text{ cm/s} \approx 0.02 \text{ m/s}$$

$$v = K_g i, \qquad i = \frac{v}{K_g} = \frac{0.02}{0.02} = 1$$

i.e. the cone of depression has a slope of 45° which is critical.

Factors Affecting Permeability

The coefficient of permeability (K) is the rate of flow per unit cross-sectional area under unit hydraulic gradient (at a specified temperature) and is usually expressed as lpd/m² or m/sec. It has the dimensions of velocity (L/T). The permeability depends upon the grain size distribution, porosity, shape and arrangement of pores, properties of the pore fluid and entrapped air or gas and can be expressed as

$$K = CD^2 \frac{\gamma_w}{\mu} \frac{e^3}{1+e} \tag{4.11}$$

where C=a constant; D=effective size of the formation material (aquifer); e = void ratio; γ_w = unit weight of water at the flow temperature and μ = viscosity of water at the flow temperature.

The intrinsic or specific permeability (k) of a water bearing medium is given by

$$k = CD^2 \tag{4.12}$$

where the constant C summarises the geometrical properties of the porous medium and k has the dimensions of L^2. k when expressed in cm² is usually extremely small, so that Darcy has been adopted as a more practical unit.

$$1 \text{ darcy } (k) = 0.987 \times 10^{-8} \text{ cm}^2$$

Coefficient of permeability, $K = k \frac{\gamma_w}{\mu}$ (4.13)

1 darcy $(K) = 0.966 \times 10^{-3}$ cm/sec (for water at 20°C)

Since $\gamma_w = \rho g$, where ρ is the density of water, $K \propto \frac{1}{\mu/\rho}$ or $\frac{1}{\nu}$, i.e. the permeability at different temperatures varies inversely as the respective kinematic viscosities. Hence if permeability at laboratory temperature K_L (temperature of determination) is known, the permeability at any other temperature K_t can be determined from the equation

$$K_t = \frac{K_L \nu_L}{\nu_t} \tag{4.14}$$

While determining permeability (K), the water temperature has to be noted and K has to be reduced to a standard temperature of 20°C or 27°C

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(f) the volume of water released from storage from the entire aquifer due to a unit decline of piezometric head

[1—c; 2—b; 3—b, c ; 4—c ; 5—b, d, e]

QUESTIONS

1. In Chandrapalem basin, AP, India, consisting of 22 km² of plains, the maximum fluctuation of ground water table is 3 m. Assuming a specific yield of 16% what is the probable ground water storage? (10.56 M.m²)
2. Calculate the fresh water flow in a coastal aquifer extending to a length of 40 km along the coast, assuming an average permeability of 40 m³/day/m², average thickness of aquifer of 20 m, and the piezometric gradient at 5 m/km. (160,000 m³/day)
3. An aquifer averages 50 m in thickness and is 100 ha in area. Determine the volume in ha-m of water available if
   (a) the aquifer is unconfined and is completely drained
   (b) the aquifer is confined and the piezometric head is lowered from 30 m to 10 m above the aquifer
   (c) the aquifer is confined and the piezometric head is lowered 55 m which brings the water table 25 m below the confining layer.

   Assume $S_y = 15\%$, $S = 2 \times 10^{-4}$

   (750, 0.4, 375.6 ha.m)
4. (a) State Darcy's law and its limitations.
   (b) Discuss briefly the three regions that may be distinguished in flow through porous media. Write down the corresponding equations that are applicable.
   (c) A fully penetrating well is pumped at a constant rate of 1020 m³/hr from a confined aquifer of thickness 30 m and average grain diameter 1 mm. What is the domain around the well for which Darcy's law is applicable? Assume that Darcy's law is valid up to $R_e = 6$ and $\nu_{water} = 1$ c St ($r > 25$ cm)
5. In a homogeneous isotropic confined aquifer of constant thickness of 20 m. effective porosity of 20% and permeability of 15 m/day, two observation wells 1200 m apart indicate piezometric heads of 5.4 m and 3.0 m, respectively, above m.s.l. Assuming uniform flow, average grain diameter of sand 1 mm and ν water $= 0.01$ cm²/sec, state
   (a) whether Darcy's law is applicable?
   (b) what is the average flow velocity in pores?

   (Yes, because Re < 1, 15 cm/day)
6. Distinguish between 'ground surface contours' and 'water table contours'. Explain how the water table contours are prepared and state their uses.
7. During a falling head permeability test on a soil sample of 10 cm diameter and 20 cm length, the head in the stand pipe of 2cm diameter dropped from 50 cm to 25 cm in 2 min. Determine the premeability of the sample.

   (4.56×10^{-4} cm/sec)
8. During a constant head permeability test on a soil sample of 7 cm diameter and 15 cm length, 90 cc of water was collected in 2 min under a constant head of 50 cm. Determine the permeability of the sample.

   (5.86×10^{-3} cm/sec)

Monthly water levels (below MP in metres) in five observation wells located in an area of 260 km²

Obs. well No.	Polygon area (km²)	Jan.	Feb.	March	April	May	June	July	August	Sept.	Oct.	Nov.	Dec.	Jan.
1.	39	3.60	4.05	4.12	4.57	4.80	4.95	5.02	4.80	4.42	4.20	3.90	3.30	3.00
2.	65	1.80	1.95	2.10	2.32	2.77	2.92	3.00	2.62	2.47	2.40	2.25	2.10	2.25
3.	52	2.70	2.92	3.22	3.37	3.52	3.60	4.20	3.60	3.37	3.15	2.77	2.40	2.55
4.	39	1.50	1.80	1.92	2.17	2.40	2.70	3.15	2.85	2.62	2.32	2.02	1.80	1.95
5.	65	1.35	1.65	1.87	1.80	2.40	2.77	3.22	3.00	2.62	2.18	1.80	1.42	1.50

(Ans: −14, −23.4, −32.4, −51.2, −62.3, −79.5, −51.40, −47.4, −34.4, −18.9, −1.8, −5.2M. m³, cumulative)

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